SCHOOL OF CIVIL ENGINEERING

# JOINT HIGHWAY RESEARCH PROJECT

JHRP-78-7

## SOIL COMPACTION SPECIFICATION PROCEDURE FOR DESIRED FIELD STRENGTH RESPONSE

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PURDUE UNIVERSITY

INDIANA STATE HIGHWAY COMMISSION

Interim Report

SOIL COMPACTION SPECIFICATION PROCEDURE

FOR

DESIRED FIELD STRENGTH RESPONSE

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| TO: | J. F. McLaughlin, Director<br>Joint Highway Research Project | June 1, 1978 |
| | | Project: C-36-5M |
| FROM: | H. L. Michael, Associate Director<br>Joint Highway Research Project | File: 6-6-13 |

The attached Interim Report is submitted on the HPR Part II study titled "Improving Embankment Design and Performance". This is Interim Report No. 4 and is titled, "Soil Compaction Specification Procedure for Desired Field Strength Response". It has been authored by Mr. John T. Price, Graduate Instructor in Research on our staff, under the direction of Professor A. G. Altschaeffl.

This report addresses the procedural development of a soil compaction specification which insures a desired field strength response from common compaction practices. The findings are the result of sampling and testing of a silty clay soil test pad field-compacted as part of an ISHC contract at Anderson, Indiana; those results were correlated with those of Interim Reports No. 2 and 3 on the laboratory testing results from a similar soil. Research continues on this project to develop similar capabilities for other soils and properties.

This report is submitted in partial fulfillment of the objectives of this study, and essentially completes the original phase. Following acceptance by the JHRP Board it will be forwarded to ISHC and FHWA for review, comment and similar acceptance.

Respectfully submitted,

Harold L. Michael

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Associate Director

HLM:ms

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Interim Report

# SOIL COMPACTION SPECIFICATION PROCEDURE

# FOR

# DESIRED FIELD STRENGTH RESPONSE

by

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Graduate Instructor in Research

Joint Highway Research Project

Project No.: C-36-5M

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Prepared as Part of an Investigation

Conducted by

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16. Abstract

A glacial silty clay, previously studied in the laboratory, was compacted in a field test pad to determine what variables control density and strength and their variabilities. Correlation was attempted to provide a prediction process for the field result. Additionally, a procedure was identified by which quality assurance and design engineers can interpret and write compaction specifications that insure a desired field strength.

Strength is controlled by water content, density, and compactive effort. Dry density is controlled by water content, compactive effort, and the interaction between them. The quantitative influence of each constituent variable varied with type of equipment.

Variabilities of both density and strength are significant. Predictions are possible if the magnitude and a range in magnitude of each constituent variable are known. Control of the compaction process and good soil homogeneity reduce the variabilities.

A procedure is presented for this soil which develops a computer tabulation of the variables to allow an estimate of field compacted strength knowing inspection test results. Also developed is a procedure to allow the design engineer to directly determine his compaction specification to insure a desired strength and its variability.

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# TABLE OF CONTENTS

## LIST OF TABLES

# LIST OF FIGURES

Figure Page

## LIST OF SYMBOLS AND ABBREVIATIONS

| | |
|---|---|
| AASHTO | - American Association of State Highway and Transportation Officials |
| AASHO | - American Association of State Highway Officials |
| CBR | - California Bearing Ratio |
| ISHC | - Indiana State Highway Commission |
| PCF | - Pounds per cubic feet |
| PSI | - Pounds per square inch |
| $R^2$ | - Coefficient of determination |
| r | - Correlation coefficient |
| $G_s$ | - Specific gravity |
| SFR | - Sheepsfoot roller |
| RTR | - Rubber-tired roller |
| A.C. | - As-compacted |
| S. | - Soaked |
| $\bar{x}$ | - Mean value of x |
| $\bar{y}$ | - Mean value of y |
| $\sigma^2$ | - Statistical variance |
| $\sigma$ | - Standard deviation |
| $\sigma_\beta^2$ | - Between-treatment variance |
| $\sigma_\omega^2$ | - Within-treatment variance |
| w | - Water content |
| $\bar{w}$ | - Mean water content of data used for prediction model development |

$E$ - Compactive effort

$\bar{E}$ - Mean compactive effort of data used for prediction model development

$\gamma_d$ - Dry density

$\bar{\gamma}_d$ - Mean dry density of data used for prediction model development

$q_u$ - Unconfined compressive shear strength

$\hat{\gamma}_d$ - Expected dry density

$\hat{q}_u$ - Expected unconfined compressive shear strength

$V(w)$ - Variation in the water content

$V(\hat{\gamma}_d)$ - Expected variation in the dry density

$V(\hat{q}_u)$ - Expected variation in the unconfined compressive shear strength

$(\hat{q}_u)_m$ - Minimum expected unconfined compressive shear strength

# NOTES OF CLARIFICATION

Strength unless noted otherwise means the unconfined compressive shear strength.

Water content unless noted otherwise means the water content at the time of compaction or the molding water content.

Density unless noted otherwise means the dry density of the soil immediately following compaction.

Energy and compactive effort unless noted otherwise mean the number of roller passes.

The symbol ' ^ ' over any variable denotes the variable magnitude derived from the regression analysis. A variable without the symbol ' ^ ' denotes an observed or measured magnitude of the variable.

## HIGHLIGHT SUMMARY

This is the fourth interim report for this project. The objective of this work is to evaluate the variables controlling the behavior of the field compacted silty clay previously tested in the laboratory. With the variables understood, a technique is desired for the prediction of in-service properties.

A field test pad was constructed using both a sheepsfoot and a rubber tire roller. Samples were taken and tested in unconfined compression in both as-compacted and soaked condition. Statistical analyses were made to determine the most useful predictive models for strength and dry density. As a result of using reasonably routine construction procedures on the test pad, only wet-of-optimum water content results could be studied. These results were compared with those of two earlier project interim reports, JHRP 76-28 by Essigmann and JHRP 77-8 by Scott.

Regression models indicated that, in general, strength and density are influenced by water content, compactive effort and dry density in a variety of functions and interactions. In some cases (e.g. as-compacted strength of sheepsfoot compacted soil), water content showed the largest influence upon the result. In other (as-compacted dry density, rubber-tire roller), compactive effort and square of water content were significant influences.

Variability of the strength magnitude was found to be sizeable but predictable. Each regression model showed a characteristic variability envelope determined by knowing the magnitude and variability of each variable in the model. Larger uncertainty in the constituent variables causes a less precise forecast of the strength.

Using graphic superposition, a procedure was developed by which the design engineer can account for both the desired strength (as-compacted or soaked) and its variability in the development of compaction specifications. The procedure indicates how much compaction to specify to assure an expected strength with a selected variability.

Additionally, a computer tabulation format was prepared to list independent compaction variables and their uncertainty, the expected strength response, and the possible range of the expected strength. From such format quality assurance engineers are able to estimate the performances of the compacted soil.

A user's guide was developed which indicates how to use the results of this study; this guide is entitled "Application of Results". Complementary text discusses how the user's guide was developed so that similar guides for other soils and/or equipment can readily be developed.

# INTRODUCTION

Compacted soil is one of the most important items in the engineer's warehouse of materials. It is, in fact, the primary construction material for earthen dams and highway embankments. Since the goal of the design engineer is to provide a safe structure at the least cost, the need exists to understand the compacted soil's engineering properties such as soil structure, permeability, compressibility, rate of consolidation, swelling characteristics and shear strength. As an embankment's stability may be governed by the shear strength of the compacted soil, this report investigates this strength property, its magnitude and variability.

During an embankment design, the engineer must select or estimate the expected soil strength as well as devise specifications to insure this strength's achievement. One method to estimate the expected strength is to construct a special fill section using a range of compaction processes and then test samples from the soil mass after each process. A test pad so constructed with the associated costs of field sampling, laboratory testing and analysis is not economically feasible for most projects. The problem is compounded if more than one soil type is used within the proposed embankment. Therefore, the design engineer must infer the strength behavior of field-compacted soils from laboratory-developed compaction curves. As this inference process may not be the most

desirable, this report will attempt to develop a more rational method of predicting the field post-compaction strength response from laboratory tests.

To accomplish the prediction model intent, a special test pad was constructed from which samples were taken and tested for unconfined strength. As the water content, dry density and compactive effort were measured or counted for each sample, relationships were then developed for dry density as a function of water content and compactive effort and for strength as a function of water content, compactive effort and dry density. Both the as-compacted and soaked soil conditions were investigated. Once these relationships were developed for the field-compacted samples, they were compared to the results obtained by Essigmann (1976) and Scott (1977) who developed similar relationships for a laboratory compacted soil (soil similar to that used in the test pad) tested in the as-compacted and soaked conditions, respectively. A prediction method was attempted from the comparison of the field- and laboratory-compacted soil relationships.

An inherent variability is expected in the results of tests performed on any compacted embankment because the soil type and soil condition are not uniform and because compaction processes and sampling and testing programs can not be exactly duplicated. Therefore, the dry density and strength characteristics of compacted soil are expected to vary within some definitive range. This report investigates the variation found in the field-compacted soil and attempts to develop a method to predict its magnitude. This is to

allow a design engineer to predict not only the expected average unconfined soil strength, but also the variation from this average strength that can be expected. Should this be accomplished, the engineer will have a method for developing a compaction specification for subgrades and low embankments that assures a minimum soil strength; the writer will explain a method by which the variability may then be accounted for in the compaction specification.

From this study two results are desired. The first result is a user's guide that enables a quality control engineer to estimate the strength and its associated variability from inspection tests. In effect, this eliminates the necessity of obtaining undisturbed samples during construction or of using in-situ strength testing methods. This guide is intended to provide the design engineer with a procedure to establish compaction specifications that insure minimum strength requirements. The other desired result is to explain how the user's guide was developed. This provides other researchers a procedure from which complementary user's manuals may be established for other soil types and compaction equipment.

# PROJECT PURPOSE

## Strength-Density Assumptions

Design engineers of fill sections and embankments are concerned with developing compaction specifications that are sufficient to insure adequate strength behavior of the compacted soil mass. Three general types of specifications are currently in use. The first type requires a desired end result. Most end result specifications require the post-compaction field density to be some predetermined percentage of the maximum density derived from a standard laboratory compaction test. Also, a range in permissible water contents is usually stated. The advantages associated with this format type are two-fold:

1) Contractors are given the freedom to choose the most economical equipment and compaction process that render the desired density within the specified water content range.

2) Density measurements usually offer accurate densification determinations as the specific gravities of different soils rarely fluctuate significantly.

The Highway Research Board (1952) lists the disadvantages associated with using this method:

1) Expensive field-test determinations of water content and density must be made.

2) An accurate measure of the soil densification can not be made if unknown and unlike specific gravities do exist.

3) An inappropriate laboratory control-density value may be used as the specification criterion if the field soil is identified incorrectly.

The second specification format requires the use of a particular compaction process. Equipment type and operation, lift thickness and number of coverages or passes are partially or wholly regulated. Therefore, the control of the compaction process remains with the engineer, whose resourcefulness and experience determine the economical success or failure of the project's compaction phase. However, the contractor's experience and ingenuity in lowering the compaction cost is forfeited.

The third format is a combination of the first two. Density, water content, lift thickness, equipment type and equipment use are all specified. This constitutes the most rigid of the specification types and requires a high level of competence of the engineer for an economical design result.

All three compaction specification formats are similar in that they are either directly based upon or are concerned with the resulting field-densification relative to the density obtained from a standard laboratory compaction test on the same soil. None directly address the soil strength property, even though the strength property is often the primary reason for compacting the soil. One reason for specifying density rather than strength is economics. The cost of an inspector making a few density control-test determinations is

much less than to conduct a field sampling and laboratory testing program for shear strength determinations. Another reason for regulating density rather than strength is that little research has been undertaken to relate the field soil conditions and compaction processes directly to the resulting strength. A greater volume of research on field compacted soils has been directed towards the determination of the resulting density. Similarly, an enormous quantity of research has been published relating the soil conditions and compaction processes to both the density and shear strength results of laboratory compacted soils. As a result of these factors, the field shear strength resulting from any of the specification formats, is usually inferred from the measurement of the density.

This inference process may not be the most desirable because it is based on three assumptions whose applicability varies among soil types, soil conditions and compaction processes. The first assumption necessitates that strength varies directly with density for a given water content. The Highway Research Board (1952) states that for a given soil (no qualification statement made), an increase in strength is expected from an increase in density. Seed, Mitchell and Chan (1960) presented evidence showing that a decrease in the as-compacted strength is not expected with an increase in density for a given water content if the strength is defined at both moderate confining pressures and large strains. Their work encompassed a variety of soil types and methods of compaction. However, the relationship between density and strength varied for each water content, method of compaction and soil type. Using a 1 kg/sq.cm confining

pressure, Casagrande and Hirschfeld (1962) found that the Canyon Dam Clay, tested unconfined and undrained, exhibited a positive correlation relationship between strength and density. Seed and Monismith (1954) found similar conclusions by interpreting the results of the work completed at the Waterways Experiment Station (1949). They presented evidence that for a silty-clay soil compacted in the laboratory by a kneading compactor, the as-compacted strength increased as density increased for a given water content. However, for a high water content (wet of the optimum water content for all but the lowest compactive effort level), the strength decreased with an increase in density. Again, the strength-density relationships differed for each water content.

These investigations found, however, that at low-strain failure criterions, the strength may or may not increase with an increase in density. Seed and Chan (1961) found that kneading compaction samples of a silty-clay soil tested in the as-compacted condition, showed that strength increased with increasing density up to a certain point and then decreased with increasing density. A 5 percent strain criterion was used for determining failure for these samples. Seed, Mitchell and Chan (1960) also reported that at a 5 percent strain failure criterion, the soaked strength of a kneading compacted silty-clay increased and then decreased for increasing densities. These results suggest the first necessary assumption is of doubtful validity.

The second assumption in inferring strength from density is the strength curve must be similar both in shape and in orientation to the density curve corresponding to the same compaction process.

Useful correlations of the type now assumed in the typical compaction specifications (Indiana State Highway Commission, 1974) can be made only if the strength curve has a characteristic peak as does the density curve. Little or no inference could be made if the strength curve was similar to a log spiral, parabola, hyperbola, etc. Furthermore, should a characteristic peak in the strength curve exist, its occurrance must be very near the optimum water content to accurately relate strength to density. The literature is inconclusive regarding both of these points.

Horonjeff (1954) presented evidence that soaked CBR strength curves for a silty-clay compacted by the impact method closely resemble the corresponding dry density curves. The CBR curves shown have not only the characteristic peaks, but also the occurrence of the peaks at approximately the optimum water contents for the various compactive effort levels. Seed, Mitchell and Chan (1960), using a kneading compactor but also working with a silty-clay, found that the strength curves sometimes peaked and other times did not, depending upon the compactive effort level. The evidence pertained to soil tested in the as-compacted condition using the criterion that strength was the stress required to cause either a 25 percent or 5 percent strain. They also found that the 5 percent strain failure criterion CBR curves characteristically peaked near the optimum water content for soil soaked before testing. The Highway Research Board (1952), using a Proctor Penetration Needle found the penetration resistance curve concave upward throughout the range of water contents in which the density curve peaked.

One of the most comprehensive studies on the relationship between density and strength was performed at the Waterways Experiment Station (1949). The work included laboratory impact and static compaction methods and field rolling of silty clay and clayey sand soils. CBR tests were used for all strength measurements. The impact compaction data for the silty clay and clayey sand showed the same curve trends as did the kneading compaction samples of Seed, Mitchell and Chan (1960). However, the statically compacted laboratory samples showed no peaks in their CBR curves for either the as-compacted or soaked soil conditions. For the field compacted silty clay, the same general trends continued; the soaked sample CBR curves showed similar shapes and orientations as did the density curves and the as-compacted CBR curves showed little consistency in either shape or orientation. The equipment used consisted of a rubber-tired roller and a sheepsfoot roller, each of which was ballasted to produce varying maximum wheel loads and foot contact pressures, respectively. For all of the literature cited herein, if peaks did exist in the as-compacted strength curves, regardless of whether the compaction took place in the field or in a laboratory, they tended to be displaced significantly to the dry side of the laboratory optimum water content.

Because the expense of obtaining good field compaction (density) curves is prohibitive for many projects, the strength derived from a field compaction process is inferred not from field-density curves but from laboratory density curves. In so doing, the assumption is made that the field strength is related in a nearly identical manner

to the laboratory derived density as is the laboratory strength. This constitutes the third assumption made in most present compaction specifications that use a percent of maximum density either directly or indirectly as a measure of strength. To make the transition between field strength and laboratory strength, two approaches are possible. The first method is straightforward: directly relate the field strength obtained from a particular soil condition and compaction process to the laboratory strength derived under similar conditions. The second method is not as direct; an assumption must be made that the field strength is related to the field density in a like manner as the laboratory strength is to the laboratory density. Then a correlation must be shown to exist between the field and laboratory compaction curves.

In an effort to establish a relationship between field strength and laboratory strength and/or density for a silty clay soil, the Waterways Experiment Station (1949) used the second approach. The results showed that the field optimum water content was 2 to 3 percentage points greater than the laboratory modified Proctor optimum although the shapes of the two compaction curves were quite similar. Of greater importance was the conclusion that the CBR values of the field compacted samples could not be predicted from the CBR values of the laboratory compacted samples even though the CBR values were taken at the same water content and density. They explained that the CBR values were not comparable because the relative positions on the compaction curves of the field and laboratory compacted samples were not similar for identical values of density and

water content. However, when using triaxial compression cells for as-compacted strength determinations, the shear strength was comparable between field and laboratory compacted samples. This similarity was found for both impact and static laboratory compaction techniques although when using higher confining pressures on the statically compacted samples, little correlation could be ascertained. Essentially identical tests were run on a clayey sand material. The results indicated that the standard AASHO compaction curve was closely similated by the field compaction curves derived by various compaction equipment and processes. Even with this similarity in both shape and orientation of the field and laboratory compaction curves, the CBR values were not found to be comparable.

As shown in the preceding discussion, inferring the resultant strength from measurements of water content and density may not be sufficient to insure the design engineer that a minimum desired strength has been achieved. It is, therefore, the intent for this research, coupled with the work performed by Essigmann (1976) and Scott (1977), to devise a better method of predicting the field strength from laboratory compaction and strength tests.

## Variation of Compaction Results

Variation in compaction results occurs regardless of the stringency of methods taken to prevent its development. As uniformity in soil strength characteristics is one criterion necessary for providing an adequate foundation for highway pavements, compaction techniques should be employed that reduce the resulting variability

as much as is economically feasible. Many quality assurance programs for compacted fills consist of taking only a few, many times just one, control tests for an extensive volume of compacted soil; if the results of such tests indicate to the engineer's best judgment that the samples tested meet the specification, then the entire fill section from which the samples came are accepted. Neither the small number of tests performed nor the standard specification in present use reflect the inherent variability found in the compacted soil mass and as such does not provide an accurate measure of the true quality of the work. Therefore, a design specification based upon both the expected results and the expected variability in those results should be implemented. Also, a quality assurance program must be initiated to provide an accurate measure of the true work quality.

## Compaction Variability

Williamson and Yoder (1968) and Williamson (1969) performed tests on a wide variety of soils compacted in the field by sheepsfoot, rubber-tired and steel wheeled rollers. Measurements of water content, density and standard maximum density were taken using then currently accepted procedures of the following field control-testing equipment: sand cone, water-filled rubber balloon and three calibrated nuclear gauges. The statistical parameter they used to measure the expected variability was either the variance $\sigma^2$, or the standard deviation, $\sigma$ (which is the square root of the variance). They concluded that there were three major factors contributing to the magnitude of the variance. These were:

1) Compaction Process Variability - This involved the inability to compact the soil in a precisely replicative manner throughout the fill section or between different fill sections. Variations in equipment type, roller operating speed, soil temperature, air humidity, lift thickness, material handling procedures and amount of compactive effort all contribute to this factor.

2) Testing Variability - Any conventional field sampling and testing program is difficult if not impossible to duplicate. Equipment accuracy and precision and the proficiency of the equipment operator largely determine the magnitude of this factor. The magnitude is usually increased if more than one operator performs the tests, if different instruments of the same kind are used or if different equipment types are employed.

3) Material Variability - Soil within any fill lift may vary to some degree due to the heterogenous conditions within the borrow area. Mixing of various soil types during the soil handling process may make the fill soil even less homogenous. Also, changing water contents within a test lift causes differing soil conditions, further contributing to the material variability factor.

As all three variability causes are interrelated, separate measurements of their magnitude have not been made. However, the relative influence of each can be categorized by applying a one-way analysis-of-variance statistical program to the data. The results of such a program divide the variance into two components.

The first component is termed the within-treatment variance ($\sigma_\omega^2$); it represents that portion of the data variability found between replicate tests on soil compacted in a similar manner to a similar condition (i.e., same water content and density). Testing variability is believed to be the major constituent of the within-treatment variability. Material and compaction process variability also influence $\sigma_\omega^2$ but to a lesser degree.

The second variance component is called the between-treatment variance ($\sigma_\beta^2$) and it denotes the data variability found between samples compacted at different locations but again, in a similar manner and to a like condition. Causes of this component, arranged in decreasing order of influence, are material variability, compaction process variability and testing variability.

The expected variability is the algebraic sum of the within-treatment and between-treatment variance components, $\sigma^2 = \sigma_\omega^2 + \sigma_\beta^2$. Most of the published results of variability determinations deal with the percent of maximum density rather than with the strength variability. However, a variability in density should indicate that a variability in the strength also exists if there is a correlation between density and strength. Tables 1 and 2, reprinted from Essigmann (1976), indicate the results of attempts to isolate the effects of material variability and the equipment portion of the compaction process variability, respectively. Of importance in these tables are the large magnitudes of the density variability which have been found and can be expected to be found using normal construction procedures.

Table 1. Effect of Soil Homogeneity on Density Variability (from Essigmann, 1976).

| Reference | Soil Type | Dry Density No. of Samples | Dry Density Mean (%RC) | Dry Density Std. Dev. (%RC) |
|---|---|---|---|---|
| Sherman, et al. (1967) | clayey silty sand, medium plasticity, homogeneous | 50 | 92.9 | 2.4 |
| | clayey silty sand, boulders to 6", heterogeneous | 50 | 90.5 | 3.1 |
| | heavy clay, sand, stone, shale, very heterogeneous | 44 | 93.6 | 5.5 |
| Jorgenson (1969) | glaciated soil area | 100 | 88.7 | 4.5 |
| | end moraine area | 98 | 89.9 | 8.04 |
| | non-glaciated area | 54 | 97.8 | 4.8 |
| Williamson (1969) | silt to silty clays, low plasticity, homogeneous | 200 | 92.4 | 5.76 |
| | low plasticity silt, to moderately plastic clays, heterogeneous | 140 | 95.5 | 6.02 |
| | highly plastic clayey sand, very heterogeneous | 138 | 96.1 | 6.33 |
| Smith and Prystock (1966) | uniform material, none greater than 3/4" | 200 | 92.86 | 2.44 |
| | fairly uniform material greater than 3/4" to occasional 6" | 200 | 90.54 | 3.09 |
| | extremely heterogeneous | 176 | 93.64 | 5.52 |

Table 2. Effect of Compaction Equipment on Density Variability (from Essigmann, 1976).

| Compaction Method | Number of Samples | Dry Density (%RC) | | Moisture Content* (%) | |
|---|---|---|---|---|---|
| | | Mean | Std. Dev. | Mean | Std. Dev. |
| Sheepsfoot Roller | 70 | 98.4 | 7.1 | -2.0 | 3.2 |
| Sheepsfoot and Pneumatic Tire Equipment** | 101 | 95.1 | 4.5 | -5.2 | 2.9 |
| Turtle*** | 40 | 93.1 | 5.3 | -3.1 | 3.3 |
| All Data | 211 | 94.9 | 5.7 | -3.5 | 3.2 |

NOTE: compacted tuff soils

* with respect to Standard Proctor optimum

** areas where sheepsfoot and rubber-tired construction equipment were operating

*** a hand operated vibratory compactor used in small confined areas

Essigmann (1976) developed a technique to predict the expected variability of both the dry density and the unconfined shear strength for a clayey silt tested in the as-compacted condition. Scott (1977) used a similar analysis to determine identical prediction capability for the same soil tested in the soaked condition. In both studies, the soil was compacted in the laboratory using the impact method. Their results indicate that the variations in dry density and strength are dependent upon both the compaction process and the soil conditions at the time of compaction. Also disclosed in their works is the first indication that interrelationships between the compaction process and soil conditions may significantly influence the resulting density and strength magnitude and variabilities. This means that the discussions of the effects of variables one-on-one with a property may be neglecting a major consideration. The magnitude of the dry density variabilities are comparable to these found in the studies mentioned above. The strength variability magnitudes were significant.

As all of the above-mentioned research studies indicate, large variabilities in the resulting dry density and unconfined strength can and do exist. These variations are attributed to varying soil conditions, testing ability and compaction processes. Only a few of the currently used compaction specifications regulate both the soil conditions and compaction process. No specification known to the author was devised to account for the expected variability found in the compacted soil. Without knowing the expected variability in either density or strength and without using this information when specifying an end result, the compaction process, or a combination

thereof, the design engineer's ability to predict the strength property of the fill or embankment is severely handicapped. Therefore, the second intent of this research is to develop a procedure for incorporating the variability into a model compaction specification.

## Quality Assurance Testing

Control tests are necessary to insure that proper compaction has been achieved. However, control tests that do not accurately measure the true quality of the work may be either expensive or dangerous, or both. Just as the density and strength variabilities should be accounted for in the compaction specifications, so must these variabilities be reviewed with regard to quality assurance control testing. A thorough study of Williamson's (1969) research will indicate the need of using a quality assurance testing program that measures the true quality of the work regardless of the variability magnitude.

The soils for all three projects in the Williamson study were to be compacted to a 95 percent AASHO T 99(A) maximum density condition as is standard with the current Indiana State Highway Commission specifications (1974). Project 1 was compacted using a towed sheepsfoot roller, a combination of a self-propelled sheepsfoot and rubber-tired roller compacted the soil of Project 2, and Project 3 received concurrent compaction by towed sheepsfoot, rubber-tire and steel wheel rollers. The compaction results are presented in Table 3, reprinted from Williamson's paper.

Table 3. Summary of Percent Compaction Results for Study Projects (from Williamson, 1969).

| | Project 1 | | | Project 2 | | Project 3 | |
|---|---|---|---|---|---|---|---|
| Category | S.C. | B. | N. | S.C. | B. | S.C. | B. |
| Number of sampling locations | 100 | 99 | 99 | 70 | 70 | 69 | 67 |
| Number of compaction determinations | 200 | 197 | 198 | 140 | 140 | 138 | 134 |
| Range of percent compaction data | 74-106 | 70-108 | 74-118 | 78-110 | 70-108 | 76-116 | 78-116 |
| Average percent compaction | 92.40 | 90.80 | 93.48 | 95.46 | 90.92 | 96.05 | 96.80 |
| Standard deviation | 5.73 | 6.63 | 7.48 | 6.02 | 7.25 | 6.33 | 6.13 |
| Percent of tests less than specification limit of 95 percent compaction | 67.0 | 74.5 | 57.5 | 43.5 | 70.7 | 50.0 | 46.2 |

S.C. = Sand Cone

B. = Balloon

N. = Nuclear

In view of the number of samples taken for each project and control-test type, the average percent compaction can be assumed to be an accurate measure of the true work quality. Project 1 would not be accepted as meeting the specifications. If a sand cone testing apparatus was employed for making the necessary control tests, Project 2 would be accepted; by using the balloon equipment, the fill would not be accepted. Project 3 would be accepted regardless of whether the balloon or sand cone equipment was used. However, any one of the projects could be accepted or rejected if only a few samples had been taken and the results of the samples had measured high or low, respectively. The opportunity for an inspector to take unrepresentative samples is large as shown in the last row of Table 3. For example, using the sand cone method for Project 3, the inspector is assured that approximately 50 percent of the samples he tested would show results that would indicate that more compaction is needed, when in fact the fill is adequately compacted. Similarly, when using a nuclear gauge for Project 1, approximately 43.5 percent of the samples tested would indicate that an inadequately compacted fill section was adequate. Results such as these clearly make it apparent that many current quality assurance programs are inadequate or inaccurate because they do not account for the variability expected within a compacted soil mass.

The South African National Institute for Transport and Road Research (1977) and Prins (1977) have presented thorough discussions of statistically planned quality assurance programs that not only reduce the probability of making the errors shown above but are

also economically feasible. Although it is not the intent of this study to develop an accurate quality assurance testing program, mention of the need is made because a compaction specification and the inspection program to measure the results of the specified compaction cannot be divorced: Both should be devised so that the desired end result is attained and can be measured.

# EXPERIMENTAL PROCEDURE

## Introduction

A special test pad was prepared by the Indiana State Highway Commission (ISHC); the soil, similar to that used by both Essigmann (1976) and Scott (1977), was compacted and then sampled. A sheepsfoot roller compacted half the test pad soil while a rubber-tired roller compacted the other half. Samples from each roller's production were tested in unconfined compression, both in the as-compacted state and in a laboratory-induced soaked condition simulating that of an in-service soil. The data from these tests were statistically analyzed to establish relationships among the water content, dry density, compaction effort and shear strength.

## Test Pad Preparation

The Indiana State Highway Commission constructed a test pad as part of its reconstruction of State Road 109 at Anderson (Madison County), Indiana. Special provisions for this test pad were developed by personnel of the Joint Highway Research Project, Purdue University, and were included within the construction contract. A plan view and typical section of the test pad are shown in Figure 1. All test pad construction and sampling occurred between late-August and mid-September, 1976.

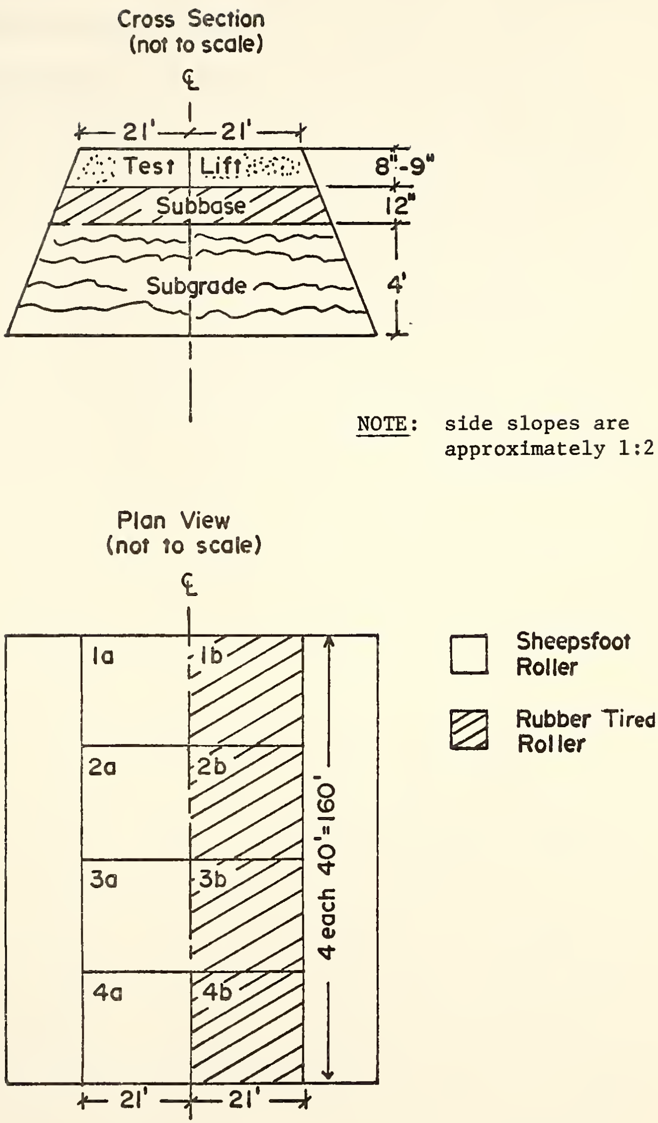



FIGURE 1 TEST PAD CONSTRUCTION

Both the subgrade and the subbase were composed of local overburden material, and each was compacted to a minimum 95 percent compaction as determined by the conventional AASHTO T99 specification (Indiana State Highway Specifications - 1974). This provided a firm foundation for the test lift. Identified earlier by Purdue personnel, a soil similar to that used by Essigmann (1976) and Scott (1977) was obtained from a designated borrow area and placed as the test lift. Care was taken to achieve a constant nine-inch loose test-lift thickness. Following the loose soil placement, a tractor-drawn disk was used to break up large clumps of the soil; unfortunately, some soil clumps remained after this disking process as the equipment was not well-suited to produce only small aggreations. It was apparent that a more rigorous mixing procedure with the disk would yield only marginal results. Figure 2 shows the equipment used for this operation.

After disking, natural water contents for each of the eight test sections (Figure 1) were evaluated with a standard model of the Speedy Moisture Content Tester manufactured by Thomas Ashworth and Co., Ltd. The natural water content for a test section was assumed to be the average of five determinations within the section. To obtain different water contents in different test sections, water was added to the lift by means of the calibrated water truck shown in Figure 3. Figure 4 is a chart, prepared for field use, by which the time of operation of the watering equipment was used to produce the desired water content. After watering, the disk was again used to blend the water and soil. Blending was difficult, particularly

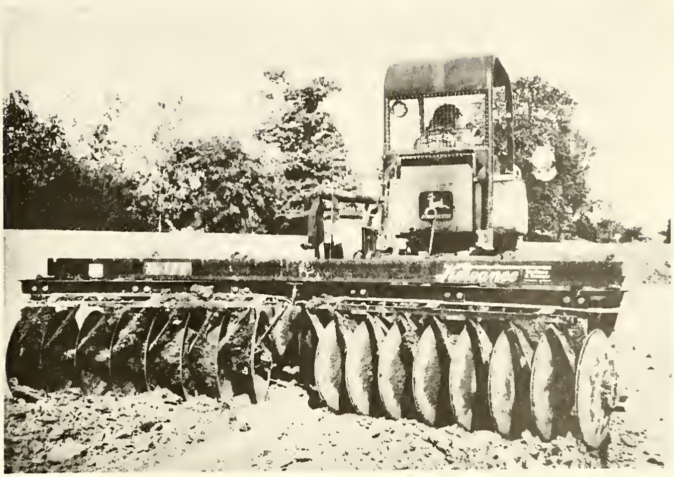


FIGURE 2 TRACTOR DRAWN DISK

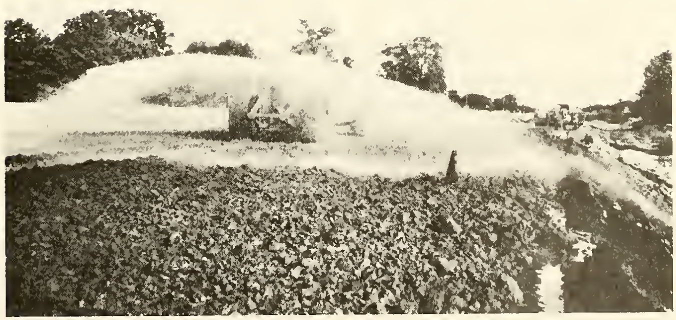


FIGURE 3 CALIBRATED WATER TRUCK

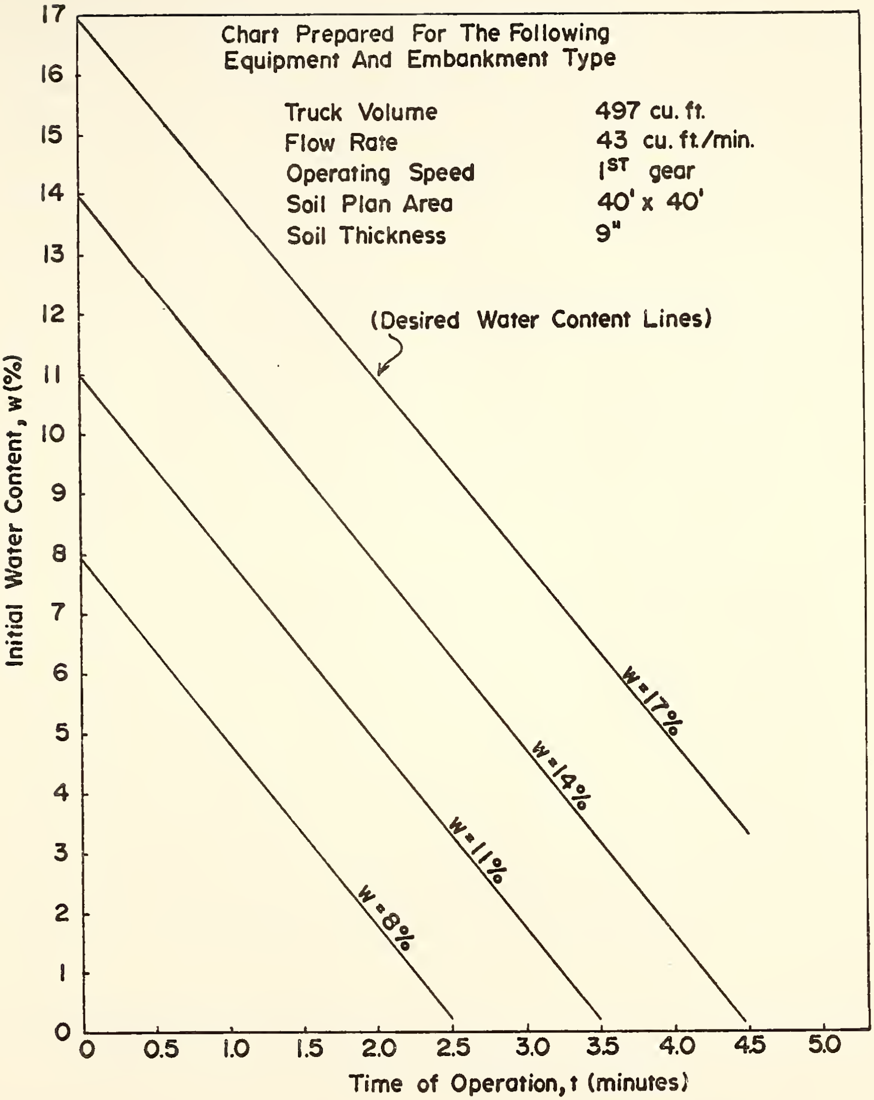



FIGURE 4 WATERING EQUIPMENT OPERATION TIME VS. WATER CONTENT

in the wetter sections, because the soil adhered to the disk blades preventing proper soil-water mixing. Each section of the test lift received at least six passes of the disk. Water content evaluations were taken after the blending process, again using the Speedy Moisture Content Tester. Large variabilities in the water content were found to exist within each test section. Since a more thorough blending was doubtful with the continued use of the test pad equipment, further efforts to produce a more uniform water content within each test pad section were not attempted. At this time, it was felt that the water content difference between the test sections and within each test section would provide data both wet and dry of the field optimum.

Due to the lack of right-of-way space and personnel necessary for the sampling process, four test lifts had to be placed and prepared; each lift was removed by scraping when its purpose had ended. The soil for each test lift came from the same borrow area and underwent preparation identical to that described above. A new subbase was constructed for each test lift to reduce the effects of differing subbase reactions caused by the continued compaction effort that resulted from the placement and compaction of each test lift.

## Soil Classification and Testing Schedule

The soil under investigation was obtained from a borrow area located within the right-of-way along State Road 109. Table 4 summarizes the identification tests performed on this soil. The table also shows the results of similar identification tests carried

out on the soil used by Essigmann (1976) and Scott (1977). The grain size distribution curve for the test pad soil is shown in Figure 5. For comparison purposes, the grain size distribution curve for the soil of Essigmann (1976) and Scott (1977) is also given.

Table 4. Soil Identification and Classification Test Results.

| Test | Anderson Test Pad Soil | Essigmann and Scott Soil |
|---|---|---|
| Liquid Limit (%) | 28 | 20 |
| Plastic Limit (%) | 18 | 14 |
| Plastic Index (%) | 10 | 6 |
| Specific Gravity | 2.73 | 2.73 |
| Unified Classification | CL | CL-ML |
| AASHTO Classification | A-4(7) | A-4(5) |
| Descriptive Name | Silty Clay | Clayey Silt |

Once the test lift soil had been prepared (each section at a different water content), the compaction equipment made a designated number of passes* over the entire test lift. The sampling process included the procurement of five samples from each test section. The location of each sample was determined using a random

*A "pass" is herein defined as one movement of the compaction equipment over an area.

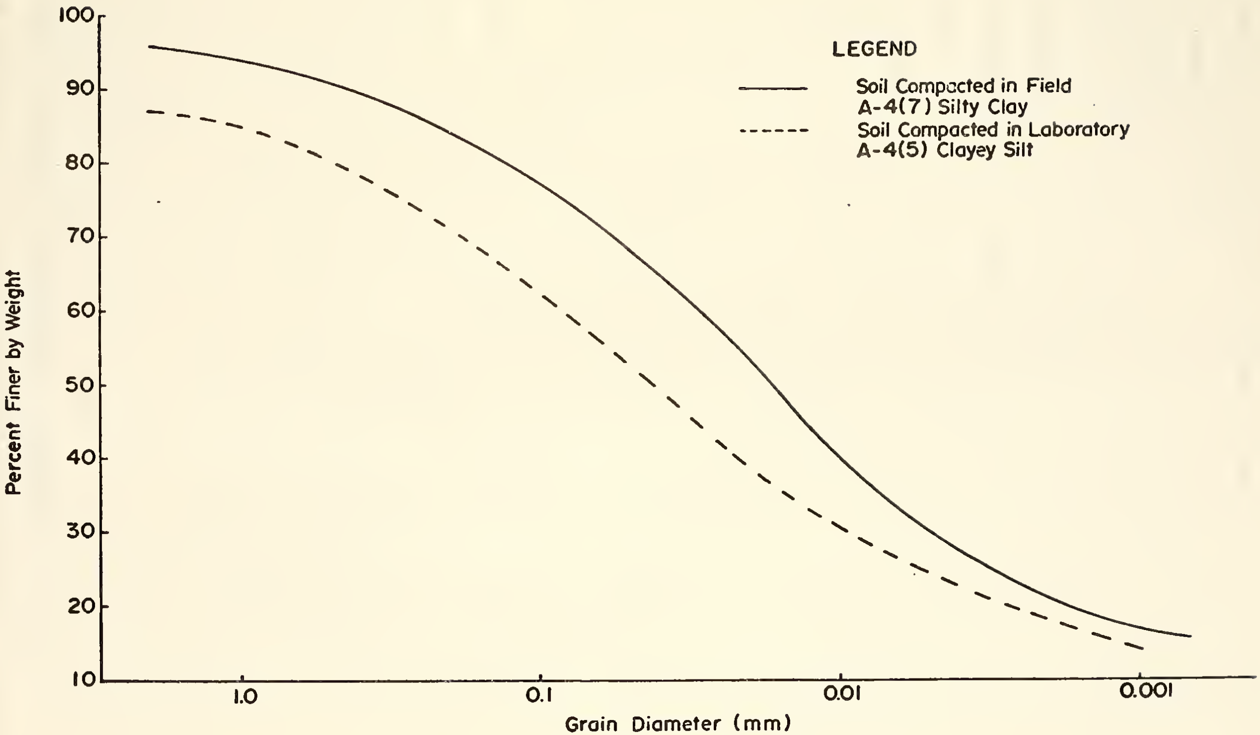


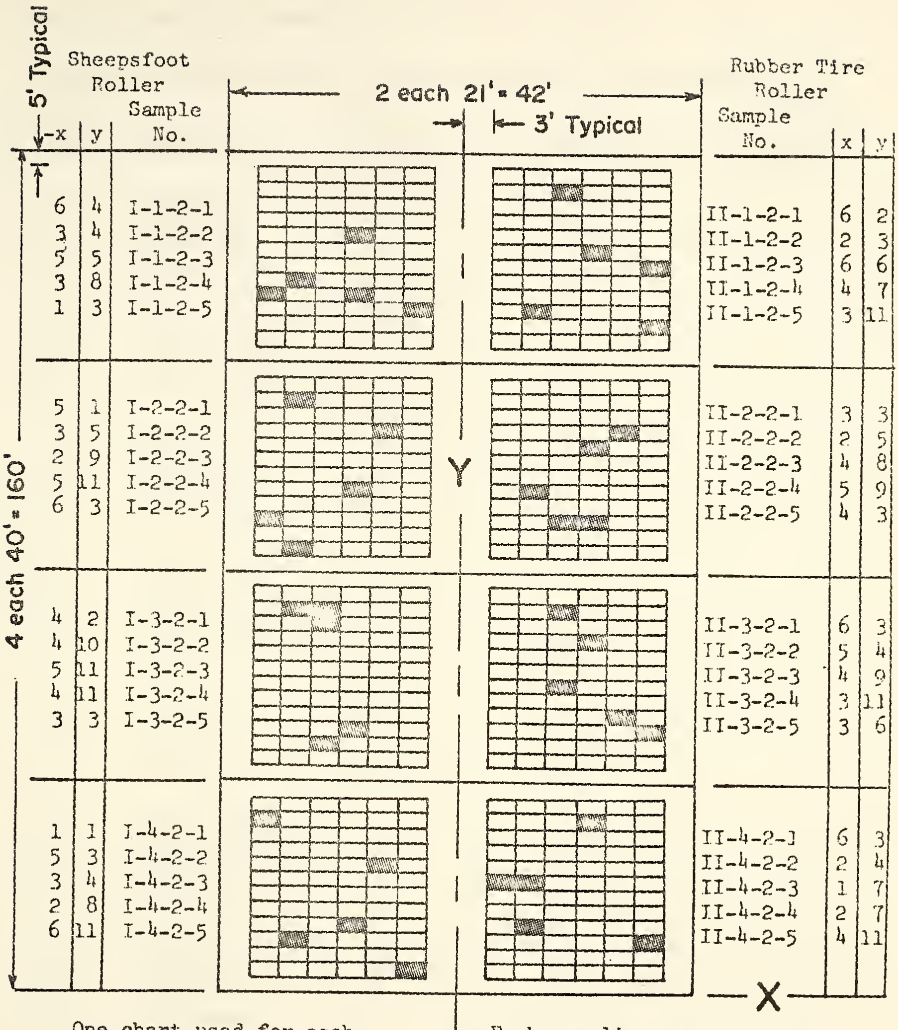

FIGURE 5 GRAIN SIZE DISTRIBUTION

number chart. Figure 6 is a typical example of the results obtained from using such a chart. For the rubber-tired roller sections, x - y measurements were made from the lower left corner of the sampling boxes for each section. The lower right hand corner was used for the same purpose for sections compacted with the sheepsfoot roller. From these points, sampling locations were found by pacing the required x - y distances. The sampling locations were different for each energy level of each test lift. In the sample identification number, the Roman numeral "I" corresponds to the sheepsfoot roller and the numeral "II" indicates compaction by the rubber-tired roller. The following number (1, 2, 3, or 4) indicates the test section and is, in turn, followed by the number of passes identification number (2, 4, 8 or 16). Finally, the last number corresponds to the actual sample number within each test section (1, 2, 3, 4 or 5).

The routine of compaction and sampling was repeated for each energy level until the sampling of the highest predetermined energy level was complete. Table 5 is a complete schedule of compaction and testing for each test lift. Originally, five samples were obtained from each test section for each compactive effort level as this would produce the required number of samples necessary for a proper statistical analysis in just two test lifts. However, the percentage of samples lost during the sampling, extruding and trimming processes was much greater than expected. Therefore, five samples were taken from each test section and the number of test lifts was increased to four to obtain a proper number of samples for testing. More than five samples could not be taken without the contractor's personnel

Sheepsfoot Roller

| x | y | Sample No. |
|---|---|---|
| 6 | 4 | I-1-2-1 |
| 3 | 4 | I-1-2-2 |
| 5 | 5 | I-1-2-3 |
| 3 | 8 | I-1-2-4 |
| 1 | 3 | I-1-2-5 |
| 5 | 1 | I-2-2-1 |
| 3 | 5 | I-2-2-2 |
| 2 | 9 | I-2-2-3 |
| 5 | 11 | I-2-2-4 |
| 6 | 3 | I-2-2-5 |
| 4 | 2 | I-3-2-1 |
| 4 | 10 | I-3-2-2 |
| 5 | 11 | I-3-2-3 |
| 4 | 11 | I-3-2-4 |
| 3 | 3 | I-3-2-5 |
| 1 | 1 | I-4-2-1 |
| 5 | 3 | I-4-2-2 |
| 3 | 4 | I-4-2-3 |
| 2 | 8 | I-4-2-4 |
| 6 | 11 | I-4-2-5 |




Rubber Tire Roller

| Sample No. | x | y |
|---|---|---|
| II-1-2-1 | 6 | 2 |
| II-1-2-2 | 2 | 3 |
| II-1-2-3 | 6 | 6 |
| II-1-2-4 | 4 | 7 |
| II-1-2-5 | 3 | 11 |
| II-2-2-1 | 3 | 3 |
| II-2-2-2 | 2 | 5 |
| II-2-2-3 | 4 | 8 |
| II-2-2-4 | 5 | 9 |
| II-2-2-5 | 4 | 3 |
| II-3-2-1 | 6 | 3 |
| II-3-2-2 | 5 | 4 |
| II-3-2-3 | 4 | 9 |
| II-3-2-4 | 3 | 11 |
| II-3-2-5 | 3 | 6 |
| II-4-2-1 | 6 | 3 |
| II-4-2-2 | 2 | 4 |
| II-4-2-3 | 1 | 7 |
| II-4-2-4 | 2 | 7 |
| II-4-2-5 | 4 | 11 |

One chart used for each energy level of each test lift (2 pass level shown).

Each sampling square represents a 2 1/2' x 2 1/2' area.

FIGURE 6 RANDOM NUMBER CHART FOR LOCATION OF TEST PAD SAMPLING

Table 5. Sampling Schedule.

| Sample Location* | Roller Type | Number of Passes | Water Content (%)** | Number of Samples |
|---|---|---|---|---|
| I-1- | Sheepsfoot | 2 | 8 | 5 |
| I-2- | Sheepsfoot | 2 | 11 | 5 |
| I-3- | Sheepsfoot | 2 | 14 | 5 |
| I-4- | Sheepsfoot | 2 | 17 | 5 |
| II-1- | Rubber-Tired | 2 | 8 | 5 |
| II-2- | Rubber-Tired | 2 | 11 | 5 |
| II-3- | Rubber-Tired | 2 | 14 | 5 |
| II-4- | Rubber-Tired | 2 | 17 | 5 |
| I-1- | Sheepsfoot | 4 | 8 | 5 |
| I-2- | Sheepsfoot | 4 | 11 | 5 |
| I-3- | Sheepsfoot | 4 | 14 | 5 |
| I-4- | Sheepsfoot | 4 | 17 | 5 |
| II-1 | Rubber-Tired | 4 | 8 | 5 |
| II-2- | Rubber-Tired | 4 | 11 | 5 |
| II-3- | Rubber-Tired | 4 | 14 | 5 |
| II-4- | Rubber-Tired | 4 | 17 | 5 |
| I-1- | Sheepsfoot | 8 | 8 | 5 |
| I-2- | Sheepsfoot | 8 | 11 | 5 |
| I-3- | Sheepsfoot | 8 | 14 | 5 |
| I-4- | Sheepsfoot | 8 | 17 | 5 |
| II-1- | Rubber-Tired | 8 | 8 | 5 |
| II-2- | Rubber-Tired | 8 | 11 | 5 |
| II-3- | Rubber-Tired | 8 | 14 | 5 |
| II-4- | Rubber-Tired | 8 | 17 | 5 |
| I-1- | Sheepsfoot | 16 | 8 | 5 |
| I-2- | Sheepsfoot | 16 | 11 | 5 |
| I-3- | Sheepsfoot | 16 | 14 | 5 |
| I-4- | Sheepsfoot | 16 | 17 | 5 |
| II-1- | Rubber-Tired | 16 | 8 | 5 |
| II-2- | Rubber-Tired | 16 | 11 | 5 |
| II-3- | Rubber-Tired | 16 | 14 | 5 |
| II-4- | Rubber-Tired | 16 | 17 | 5 |
| | | | TOTAL | 160 |

* See Figure 1. ** The water content specified is the average water content for that test section.

working overtime and without more Purdue personnel available for the sampling procedure.

## Compaction and Sampling Program

Two types of compaction equipment were used, a sheepsfoot and a rubber-tired roller shown in Figure 7 and 8, respectively. Specifications for each are shown in Table 6.

Table 6. Compaction Roller Specifications.

| Sheepsfoot Roller | Rubber-Tired Roller |
|---|---|
| Case, Model 815 | Ferguson, Model RT-2511 |
| Operating Weight 20 Tons | Operating Weight 25 Tons |
| Wheels - Tamping Foot | Tires 9:00 x 20 SWTC |
| Drum Width 38 in. | Tire Loading 4545 lbs. |
| Foot Pattern Chevron | Tire Pressure 85 psi |
| Feet/Wheel 60 | Contact Area 60.2 sq. in. |
| Feet/Row 12 | Ground Pressure 74.7 psi |
| Area/Foot 18 sq. in. | Tire Deflection 1.035 in. |
| Foot Length 7.5 in. | |

Although not measured quantitatively, the operating speed of each roller generally remained constant for a given operator throughout the compaction process. However, the two rollers were operated at different speeds and the sheepsfoot roller had more than one operator, each of whom may have operated the roller at a

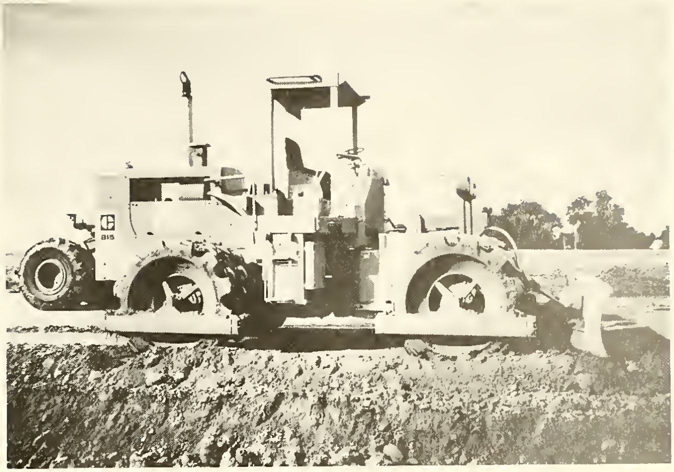



FIGURE 7 SHEEPSFOOT ROLLER


FIGURE 8 RUBBER-TIRED ROLLER

different speed. The rollers were allowed to operate in both directions. All turning, acceleration and deceleration occurred beyond the ends of the test lift, thereby avoiding unequal areas of compaction effort. Figures 9 and 10 show the test lift surface condition after two passes of the sheepsfoot and rubber-tired roller, respectively.

Each of the four test lifts used for sampling were placed and sampled on different days. The daily maximum temperatures varied from day to day but remained within a 60°F to 95°F range. Temperature variations also occurred between morning and late afternoon during any one particular day although the fluctuations were less severe. Even though Highter, et al. (1970), notes that temperature fluctuations may influence the test results, this influence must be regarded as part of the variability in construction; obviously, compaction in the field occurs over a large range of temperatures.

Thin-walled, stainless steel tubes (approximately 9 in. long by 2.0 in. outside diameter, with a 0.066 in. wall thickness) were driven into the test section to obtain samples. Figure 11 shows the driving apparatus. Each tube was lubricated with a silicone oil before driving to reduce side friction and disturbance during the driving and extruding processes. All tubes were extracted from the ground by digging. A labeling system was applied to the samples at the time of extraction. Each sample was identified according to roller type, section of test pad, compactive effort and x - y coordinates of the sample location within the test section. Samples were extruded from the tubes and each sample was placed in a plastic

FIGURE 9 TEST PAD SURFACE CONDITION AFTER 2 PASSES OF SHEEPSFOOT ROLLER; EXPECTED WATER CONTENT≈11%

FIGURE 10 TEST PAD SURFACE CONDITION AFTER 2 PASSES OF RUBBER-TIRED ROLLER; EXPECTED WATER CONTENT≈11%

FIGURE 11 DRIVING APPARATUS WITH SAMPLING TUBE ATTACHED

bag and carefully positioned in a styrofoam chest for transportation to the laboratory. Labeling tags were placed within the plastic bags and similar markings were made on the outside of the plastic bags to ensure proper sample identification.

The entire compaction, sampling and extruding process required approximately five to seven hours per lift to complete. The variation in time is due to the increased efficiency experienced by the Purdue personnel on each succeeding test lift. The first soil compaction started at approximately 9:30 a.m. for each test lift.

At the Purdue soils laboratory (Grissom Hall), each sample was trimmed, axially and diametrically measured, weighed, and prepared for the as-compacted compression test or stored for the soaked compression test. End trimmings of the samples prepared for the soaked tests were used for a determination of the original water content. If prepared for the as-compacted compression test, the sample was placed in a plastic bag and then stored for five days in a humidifier in a constant temperature room. This procedure followed that of Essigmann (1976) who had found that this produced the lowest strength-test results.

The samples used for the soaked tests were individually wrapped in cellophane then dipped in a parafin bath until a thick, continuous coating of wax was built up around each sample. Two plastic bags were placed around the samples and the samples were then stored in a constant humidity (≈ 100%) and temperature room. Lack of equipment availability necessitated the storage of these samples for a period of time varying up to eight months. A small change in the water

content (average of 0.5 percent) usually resulted from the storage.

Laboratory Testing Program

The as-compacted samples were tested in unconfined compression following the procedure of Essigmann (1976) without exception. All tests were run with a small temperature fluctuation of ±2°C from an average of 22°C. Figure 12 shows typical stress-strain curves for the as-compacted samples.

All of the field samples used to simulate the in-service conditions were soaked (≈ 1 psi confinement pressure) and tested using the procedure of Scott (1977) with three minor changes:

(1) Due to the large size of the field samples, a three-day soaking period (Scott used a two-day soaking period) was necessary to insure a sufficient degree of saturation;

(2) All the samples were tested unconfined rather than having every third sample being used for an initial water content determination. The initial water content determination at time of testing could be calculated from the measured loss of weight during storage and from the original water content determination made from the end trimmings;

(3) A single membrane was used for the soaking period. A different type of membrane was necessary for the field samples because of their size. These membranes were constructed such that small pin holes could be readily detected before soaking was initiated, and this negated the need for the use of a double membraned system.

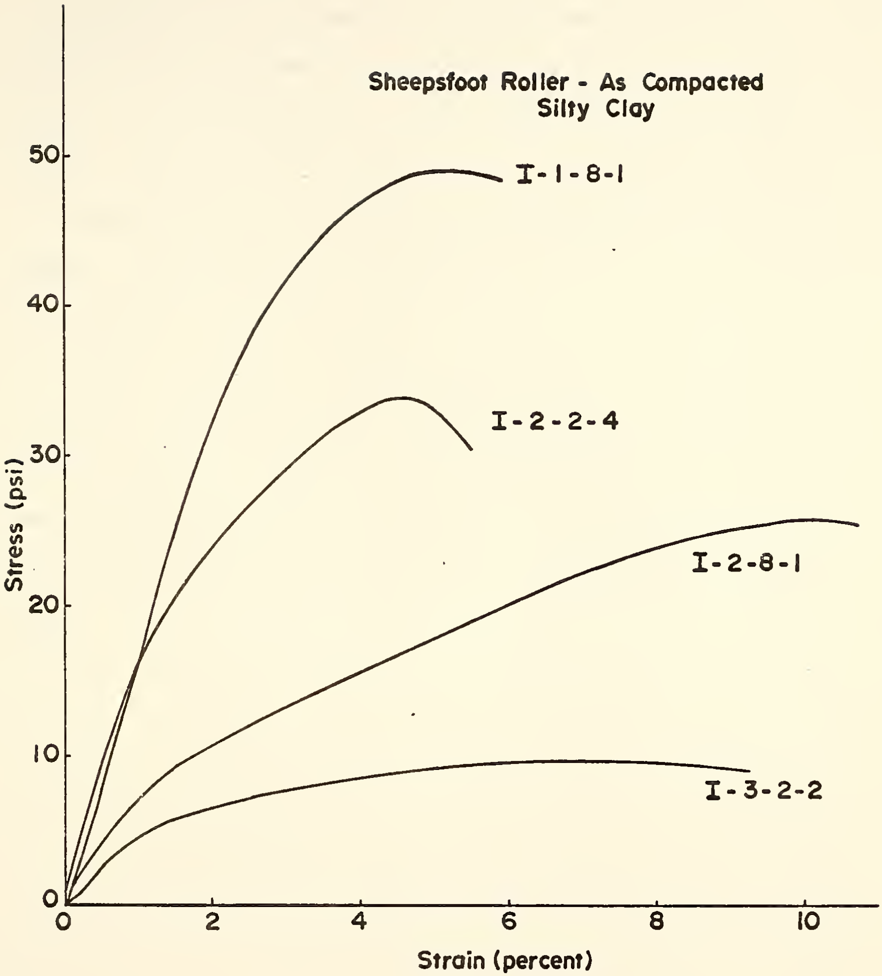



FIGURE 12 TYPICAL STRESS-STRAIN CURVES: AS-COMPACTED SAMPLES

Figure 13 shows typical stress-strain curves for the soaked samples.

Figure 15 shows the shearing apparatus and stress-strain recording system used for both the soaked and as-compacted samples. Figure 14, reprinted from Scott (1977), shows the triaxial cells and attached system used for soaking the in-service samples. Appendix A contains the complete set of results from the Anderson test pad field samples. Also included in this appendix are data sets obtained by Essigmann (1976) and Scott (1977).

The results from both the as-compacted and soaked unconfined compression tests were used to establish relationships between the shear strength of the soil with its associated variability and the compaction water content, dry density, and energy input level. The following section reviews the steps used to analyze these relationships.

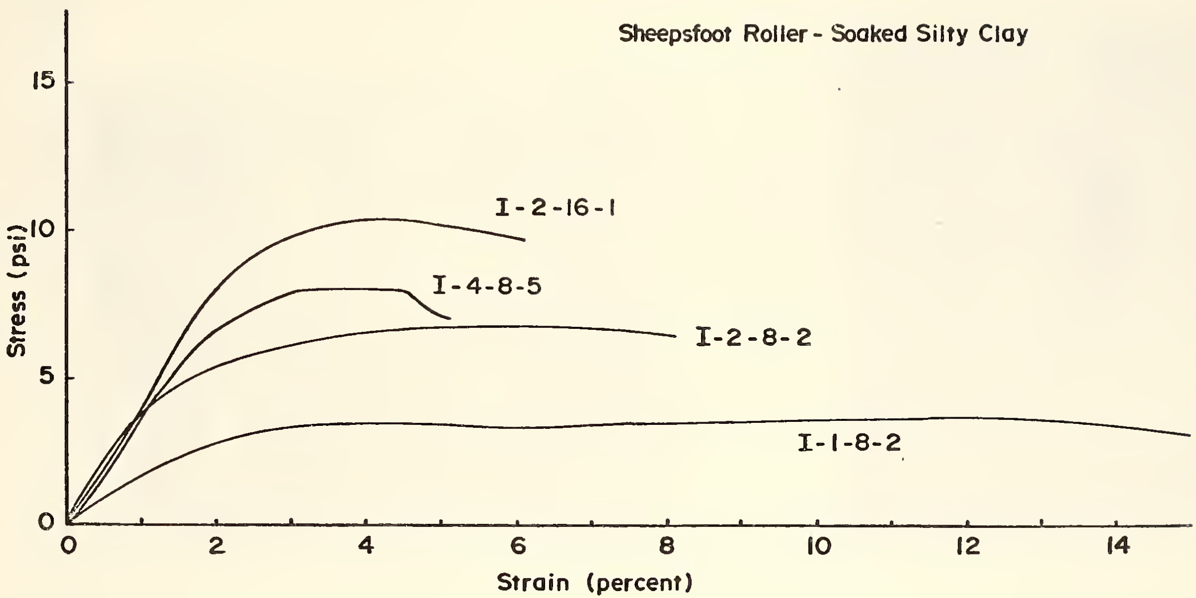



FIGURE 13 TYPICAL STRESS-STRAIN CURVES: SOAKED SAMPLES

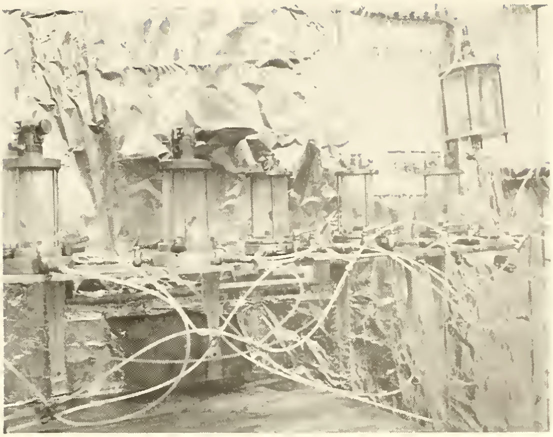


FIGURE 14 SOAKING APPARATUS (FROM SCOTT-1977)

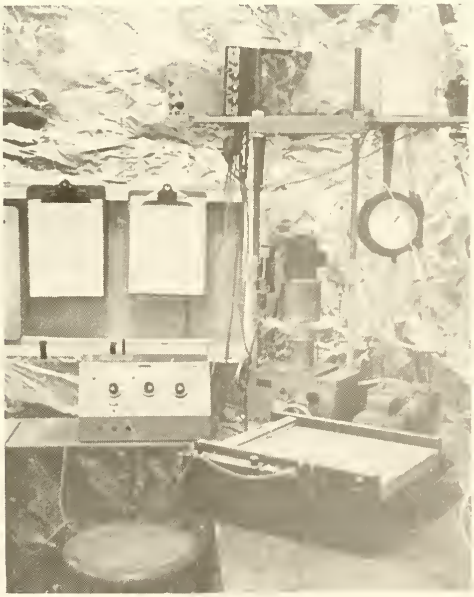


FIGURE 15 SHEARING APPARATUS AND STRESS-STRAIN RECORDING SYSTEM

# ANALYSIS OF DATA

## Introduction

The purpose of the analysis was to find the best predictive statistical model for dry density, unconfined strength and to assess the variability of the unconfined strength. This differs slightly from the work of both Essigmann (1976) and Scott (1977) whose efforts produced models of "best fit" for the corresponding data. "Best fit" models may be highly sensitive to small changes in data. Such models accurately reflect trends in one set of data, but when applied to other similar data sets, their predictive ability is substantially reduced. Models derived for predictive purposes may have less sensitivity to these small changes in data. Since the intention of this project is to develop a statistical model that allows design engineers to estimate density and strength parameters of not only the soil studied in this report but of many soils having similar compaction properties, the following analysis was developed.

A total of 168 field compacted samples were tested in unconfined compression. Of this total, 74 samples were compacted by the sheepsfoot roller; 62 of these were tested in the as-compacted condition and 12 were tested in the soaked condition. The remaining 94 samples were compacted by the rubber-tired roller; 72 of these samples were tested as-compacted and 22 were soaked before testing.

Appendix A presents the data recorded from these samples. Most of the large imbalance between the number of as-compacted and soaked samples resulted from a break-down in organization and communication between the organizer of the test pad operation and the author. A few soaked samples were lost due to bookkeeping errors and a few others were lost due to insufficient time allotted to the soaking process. These samples exhibited degrees of saturation below a 90 percent criterion arbitrarily chosen to represent the lower bound of percent saturation expected in field conditions of "saturated" soil conditions. Unfortunately, the imbalance could not be remedied at the time of its discovery.

Twelve as-compacted samples for each roller type were separated from the remaining samples before the statistical analysis was performed. These samples were used to verify the predictive ability of the regression models derived from the larger group of samples. A random number chart was used to determine which samples were to be used for the verification process. This eliminated any bias that could have developed in this selection. As a result, the statistical analysis for the as-compacted specimens was performed on 50 sheepsfoot roller samples and 60 rubber-tired roller specimens. Since so few samples were tested in the soaked condition, no soaked samples were excluded from the statistical analysis and no corresponding verification was made of the in-service predictive models.

Dry Density and Unconfined Strength Magnitude

The initial portion of this analysis isolated the prediction models that best estimated the "true" or population relationships between the magnitudes of the dependent and independent variables. Three successive steps were employed to identify these models. The first step involved the plotting of the dependent variables against each independent variable. Dry density was plotted against water content and compactive effort, and unconfined strength was plotted against water content, compactive effort and dry density. From these scattergrams, an estimation was made as to the appropriateness of including higher order terms in the predictive model. If the scattergrams showed a linear relationship with first order variables, the higher order terms were not used in further analyses. However, if a distinct linear trend was not found, all terms were considered important.

The second step of the isolation process involved the use of the Purdue computer programs DRRSQU and REGRESSION (an SPSS program developed by Nie, et al. in 1975). By using these programs, prediction models were selected and evaluated in a manner identical to that explained by Essigmann (1976). As suggested by Scott (1977), the residuals were examined to determine if they were normally distributed independent random variables. Scatter plots were made of the residuals against the ordered sequences for each independent variable. No trends in the plots were observed indicating that the residuals behaved in a random and independent manner.

The model isolation process was completed by the use of "ridge regression", a statistical technique whose application is described in detail by Marguart and Snee (1975). With the use of the Purdue computer program developed by Casella and Brnages (revised 1977), the data used earlier were re-manipulated with the addition of a designated amount of bias to the variable coefficients on successive runs for each of the prediction models already isolated. As more bias was added to the coefficients, each one was eventually driven to zero (0). The rate at which this reduction occurred determined the suitability of the associated independent variables for use in the prediction model; fast rates denoted unsuitable models. If the coefficients were extremely sensitive to the data (e.g., a small change in the data produces a large change in the coefficients), a small amount of bias added in a regression-run would cause the coefficients to quickly approach zero. Models whose coefficients exhibited this trend were not considered for further analysis.

Figures 16 and 17 are plots of RIDGE REGRESSION results for the unconfined strength of soil compacted by the rubber-tired roller and tested in the as-compacted condition. Figure 16 is a plot of the variable coefficients versus the added quantity of bias for the regression model of unconfined strength as a function of water content, dry density and the cross-product of water content and the second order of dry density. Figure 17 is a similar plot of strength as a function of water content and dry density only. As shown, the variable coefficients in Figure 16 are quickly reduced to a near-zero

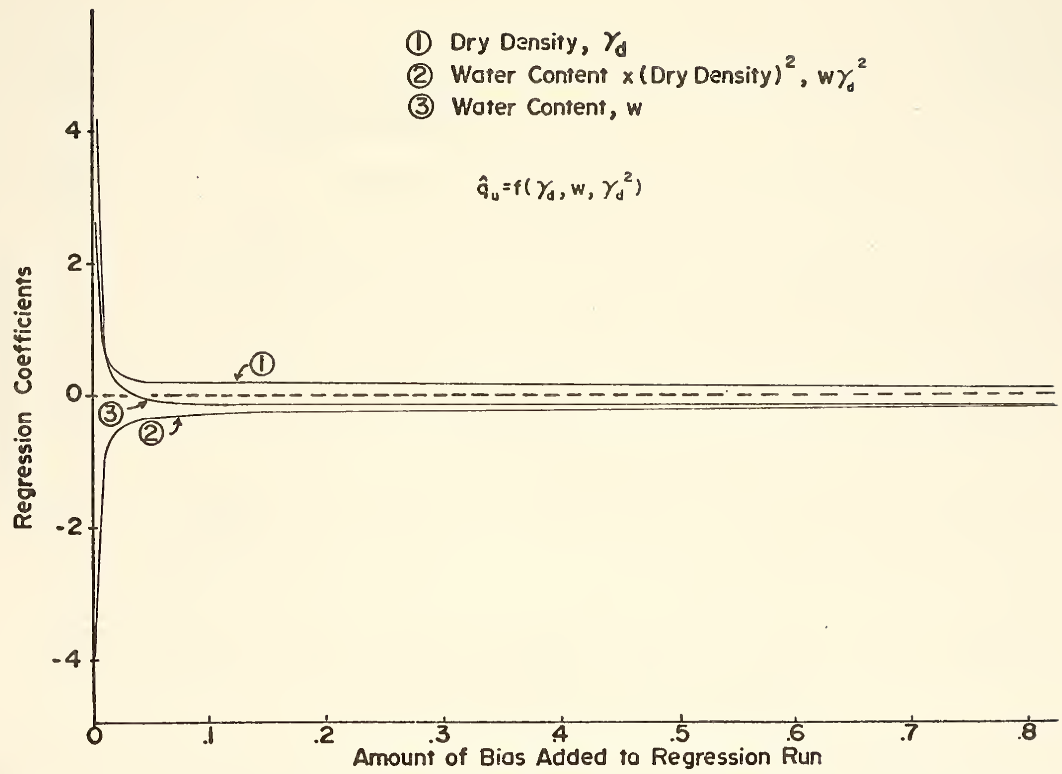



FIGURE 16 RIDGE REGRESSION PLOT OF UNSTABLE SET OF INDEPENDENT VARIABLES: RUBBER-TIRED ROLLER, AS-COMPACTED

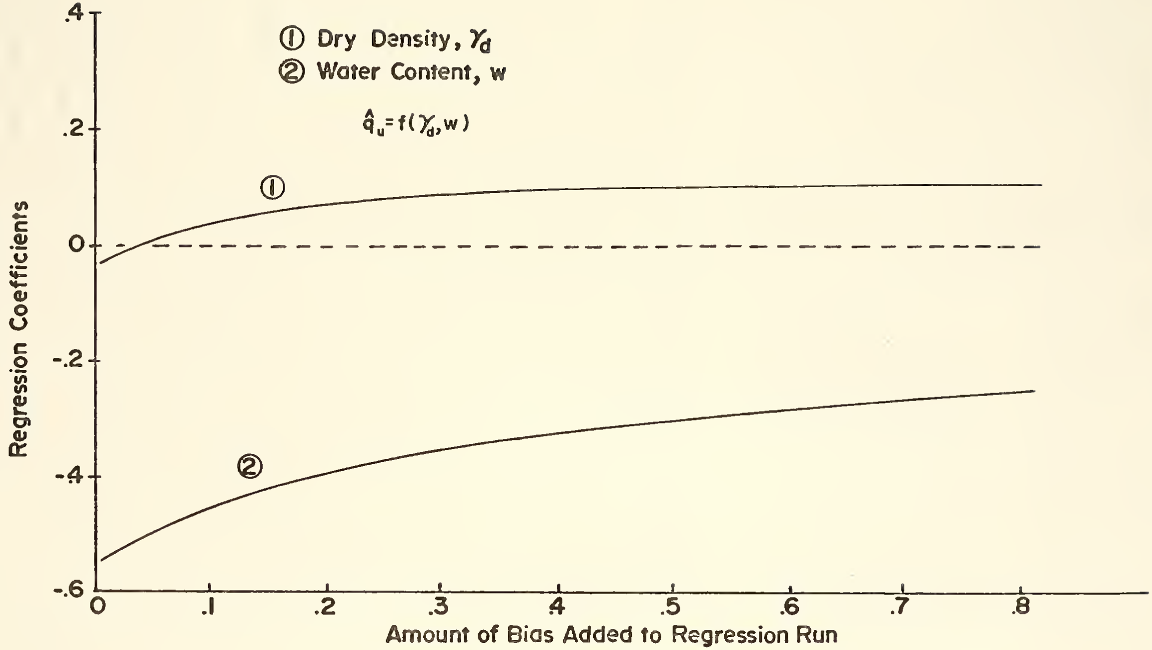



FIGURE 17 RIDGE REGRESSION PLOT OF STABLE SET OF INDEPENDENT VARIABLES: RUBBER-TIRED ROLLER, AS-COMPACTED

value (as compared to their original magnitude); the coefficients in Figure 17 are not reduced sharply by relatively large increases of added bias. The conclusion drawn from this set of plots was that the cross-product variable was not suitable to include within the strength prediction model.

Once all three isolation processes were completed, the selection of the most appropriate prediction model for each independent variable had been made. Table 7 shows the set of independent variables used for analysis. The results of the statistical analysis are presented in Table 8.

## Variability of Dry Density and Unconfined Strength

The statistical analysis developed to find the variabilities of the dry density and unconfined strength differed from that used by Essigmann (1976) and Scott (1977) for two reasons. The first reason was that an optimum water content could not be determined in the field as all field samples tested appear to be wet of the optimum water content. Therefore, 1 percent water content regions based upon the optimum water content could not be specified as was done previously. Also, Essigmann (1976) and Scott (1977) were interested in finding the variability associated with specified values of water content, dry density and compactive effort. Since all field-compacted soils showed some variability in water content and dry density, the approach was taken in this report to determine the strength variability associated with knowing a range of water contents and dry densities for a particular compactive effort. For these two

TABLE 7 VARIABLES USED IN STATISTICAL ANALYSIS

| DEPENDENT VARIABLE | INDEPENDENT VARIABLES USED IN ANALYSIS | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | $w$ | $w^2$ | $w^3$ | $E$ | $E^2$ | $E^3$ | $wE$ | $w^2E^2$ |
| DRY DENSITY | X | X | X | X | X | X | X | X |
| UNCONFINED STRENGTH | X | X | X | X | X | X | X | X |
| | | | | | | | | |
| | $w^3E^3$ | $w^2E$ | $wE^2$ | $\gamma_d$ | $\gamma_d^2$ | $\gamma_d^3$ | $w\gamma_d$ | $w^2\gamma_d^2$ |
| DRY DENSITY | X | X | X | | | | | |
| UNCONFINED STRENGTH | X | X | X | X | X | X | X | X |
| | | | | | | | | |
| | $w^3\gamma_d^3$ | $w^2\gamma_d$ | $w\gamma_d^2$ | $E\gamma_d$ | $E^2\gamma_d^2$ | $E^3\gamma_d^3$ | $E^2\gamma_d$ | $E\gamma_d^2$ |
| DRY DENSITY | | | | | | | | |
| UNCONFINED STRENGTH | X | X | X | X | X | X | X | X |

Independent variables marked above were used for both the sheepsfoot roller and the rubber tire roller in both the as-compacted and soaked sample conditions.

## TABLE 8 REGRESSION RESULTS

| ROLLER TYPE/ SOIL CONDITION | DEPENDENT VARIABLE | REGRESSION MODEL | $R^2$ |
|---|---|---|---|
| SHEEPSFOOT/ AS-COMPACTED | DRY DENSITY | $\hat{\gamma}_d = -1.398w + 133.43$ | .575 |
| | UNCONFINED STRENGTH | $\hat{q}_u = -2.058w + .585E + .353\gamma_d + 16.99$ | .419 |
| RUBBER TIRE AS-COMPACTED | DRY DENSITY | $\hat{\gamma}_d = -.0476w^2 + .169E + 122.46$ | .649 |
| | UNCONFINED STRENGTH | $\hat{q}_u = -2.224w - .155E + .0734\gamma_d + 51.91$ | .290 |
| SHEEPSFOOT/ SOAKED | DRY DENSITY | NO SIGNIFICANT MODELS | |
| | UNCONFINED STRENGTH | NO SIGNIFICANT MODELS | |
| RUBBER TIRE/ SOAKED | DRY DENSITY | $\hat{\gamma}_d = 1.529E - .0958wE + 110.75$ | .488 |
| | UNCONFINED STRENGTH | $\hat{q}_u = 1.189w + .195E + .968\gamma_d - 118.37$ | .659 |

reasons, the following statistical analysis was developed.

Two measures of the dry density and strength variabilities are of interest to this report's objectives. The standard deviation indicates the range over which the data is expected to vary. The standard error of the mean estimates the expected range of the mean value for any particular set of variables. As all regression models based on samples taken from a population are estimates of the true relationship among the variables for the population, there is an inherent uncertainty in the accuracy of the model. For linear regression, all models necessarily pass through the average values of the dependent variable and all the independent variables. The prediction accuracy is reduced the further from any one of the averages that the model is applied. Therefore, for a constant degree of confidence, the confidence interval must increase in size as the values of the dependent variables increase in distance away from their mean value. Figure 18 shows a two-dimensional example for linear regression of how the standard error of the mean and thus the expected mean value confidence interval varies for different values of an independent variable. Computations for the standard deviation and standard error of the mean are nearly identical. Therefore, only the differences in their application will be discussed in the following section.

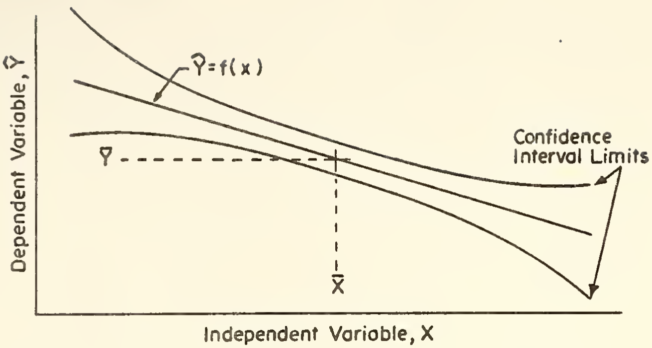



FIGURE 18 'STANDARD ERROR OF THE MEAN' VARIABILITY AS A FUNCTION OF AN INDEPENDENT VARIABLE

Zones of varying water contents were observable in many samples; the tested specimens were clearly not homogeneous as produced by field compaction. However, no measurement of the different water contents in each zone could be made and, as a result, only the average water content for the entire sample was recorded. Although each test pad section was constructed to have a nearly constant water content, a large variation in the water contents persisted. The average water content ranges for the sheepsfoot and rubber-tire roller sections were 3.3 and 3.7 percent, respectively. The assumption was made that the variations found within the samples were no greater than the variations found within the test pad section. Thus, the average range of water content found within the test pad sections was considered the expected range that must be accounted

for in predicting the dry density and shear strength variabilities.

The compactive effort was measured in number of passes and as such, no variation in the measurements is assumed. The magnitude of the dry density variability varies as the degree of confidence that is chosen. Therefore, the variability magnitude is larger for say a 90 percent confidence criterion than it is for a 60 percent criterion. This study arbitrarily uses a 95 percent confidence criterion for determining dry density variability. Thus, 95 percent of the compacted soil from which the samples were taken should have a dry density within the range bounded by the expected mean value (found in the previous section) plus or minus the appropriate factor times the standard deviation.

Once the dry density variability is evaluated, the expected variation in shear strength can be determined. Since both water content and dry density influence the shear strength magnitude and, since both vary to some degree, the evaluation of the strength variability must account for each of these variations. The amount of variation assumed for this evaluation is zero (0) for compactive effort, plus or minus the test pad sections' half-range variation for water content and plus or minus the 95 percent confidence variability found for dry density $\hat{V}(\gamma_d)_{.95}$. Table 9 presents these limits.

Table 9. Strength Variability Analysis Limits.

| Sheepsfoot Roller | Rubber-Tired Roller |
|---|---|
| E ± 0. in number of passes | E ± 0. in number of passes |
| w $\pm$ 1.65 in percent | w $\pm$ 1.85 in percent |
| $\gamma_d \pm V(\hat{\gamma}_d)_{.95}$ in PCF | $\gamma_d \pm V(\hat{\gamma}_d)_{.95}$ in PCF |

For each set of compactive effort, water content and dry density values, four computations of the strength variability were required. A value corresponding to each of the following combinations had to be calculated:

| E | w (percent) | $\gamma_d$ (PCF) | $V(\hat{q}_u)$ |
|---|---|---|---|
| E | w+1.65 or 1.85 | $\gamma_d+V(\hat{\gamma}_d)_{.95}$ | $V(\hat{q}_u)_1$ |
| E | w-1.65 or 1.85 | $\gamma_d+V(\hat{\gamma}_d)_{.95}$ | $V(\hat{q}_u)_2$ |
| E | w-1.65 or 1.85 | $\gamma_d-V(\hat{\gamma}_d)_{.95}$ | $V(\hat{q}_u)_3$ |
| E | w+1.65 or 1.85 | $\gamma_d-V(\hat{\gamma}_d)_{.95}$ | $V(\hat{q}_u)_4$ |

The four calculations were necessary because it was assumed that within this range of water content and dry density values, the two vary independently, resulting in four possible combinations of values. The largest value of $V(\hat{q}_u)$ obtained from among the four combinations was considered the largest expected value of the variability in strength for those particular values of water content, dry density and compactive effort.

The following formula obtained from McCabe (1977) was used for all standard error of the mean calculations of the field compacted soils:

$$V(\hat{q}_u) = \lambda s \sqrt{X_p{}' \, [X'X]^{-1} X_p}$$

This equation was slightly modified, as by Neter and Wasserman (1974), for calculations of the standard deviation of the data:

$$V(\hat{q}_u) = \lambda s \sqrt{1+X_p' \, [X'X]^{-1} X_p}$$

Given in Appendix B is the computer program developed to assist in the calculation process for the strength variability and an example problem using the formulas.

Although the analysis presented here was evaluated for the stated water content variation and confidence interval for dry density, the process may be adjusted to accomodate the expected conditions of the project or the requirements of the design engineer.

Results of the unconfined strength variability analysis are presented in Figures 19, 20 and 21. Since no regression model for the dry density magnitude of the sheepsfoot roller soaked samples was significant, further investigation into that variability of strength was assumed unjustified.

## Verification of Prediction Equations

To verify whether or not the regression equations obtained by the statistical analysis were good prediction models, the data from

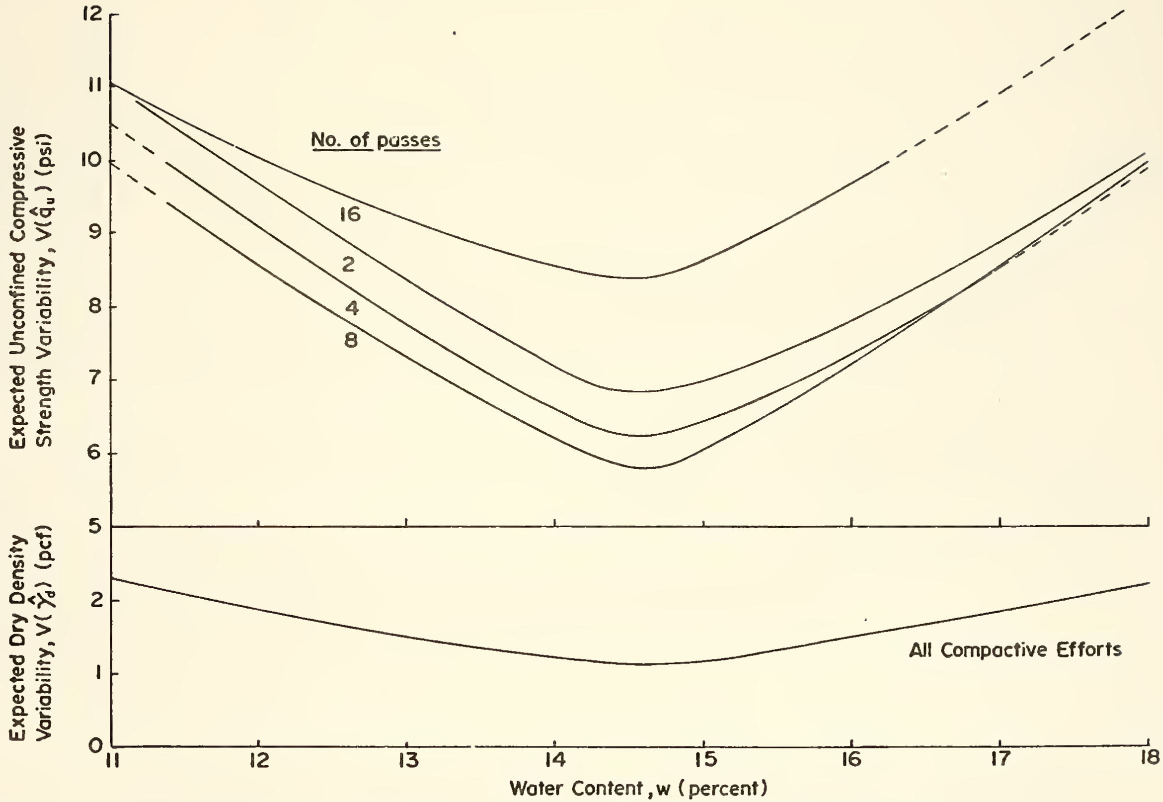



FIGURE 19 EXPECTED DRY DENSITY AND UNCONFINED COMPRESSIVE STRENGTH VARIABILITIES VS WATER CONTENT: SHEEPSFOOT ROLLER, A.C.

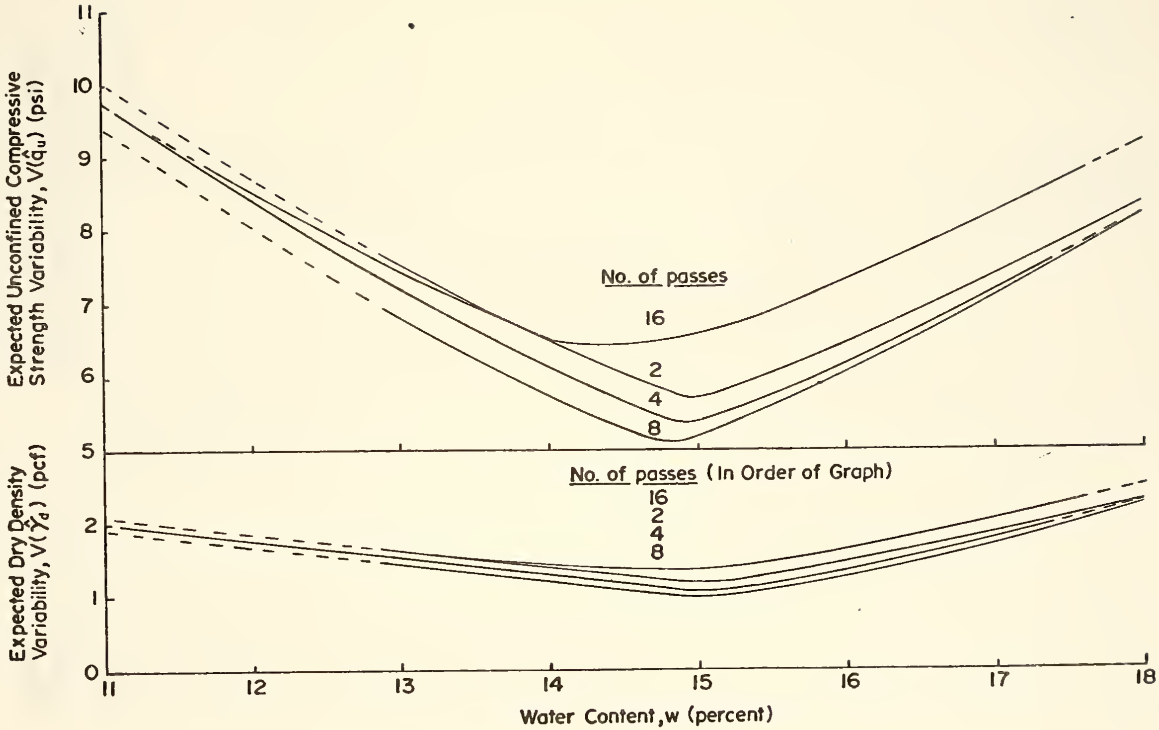



FIGURE 20 EXPECTED DRY DENSITY AND UNCONFINED COMPRESSIVE STRENGTH VARIABILITIES VS WATER CONTENT: RUBBER-TIRED ROLLER, A.C.

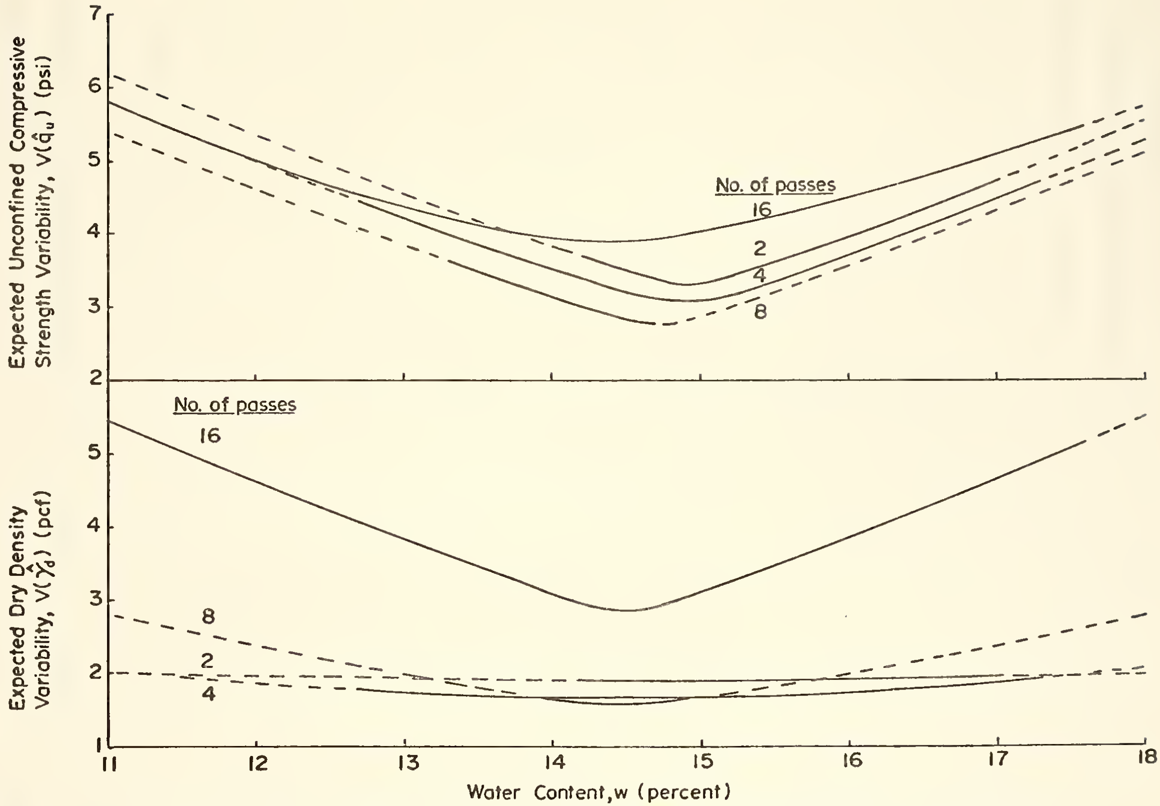



FIGURE 21 EXPECTED DRY DENSITY AND UNCONFINED COMPRESSIVE STRENGTH VARIABILITY VS WATER CONTENT: RUBBER-TIRED ROLLER, SOAKED

the 24 samples that were withheld from the analysis were compared to regression results obtained from the sample data. Dry density was calculated for each sample using the regression equation suitable for the roller type by which the sample had been compacted. Figure 22 and Figure 23 show plots of the expected values of dry density against the measured values of dry density for the sheepsfoot and rubber-tired roller, respectively. The points must lie reasonably close to the 45° line for the regression equation to be considered a good prediction equation.

The same method was used for comparing the expected and measured values of the unconfined strength. Figure 24 and Figure 25 show the two values plotted against one another for the two roller types. Again, the smaller the deviation from the 45° line, the better the model's predictive ability. Some deviation from the 45° line was expected because of the variability associated with the compaction process and the testing program. Figures 26 and 27 are plots of the expected variability against the observed variability (i.e., the difference between the measured and expected values) of the dry density. Figures 28 and 29 are plots of the expected variability against the observed variability of the unconfined strength. The expected variability was derived as discussed in the preceding section using the standard deviation form of the variability equation. Since a 95 percent confidence interval was used for all computations, approximately 95 percent or more of the samples should have observed variability values less than or equal to the expected variability.

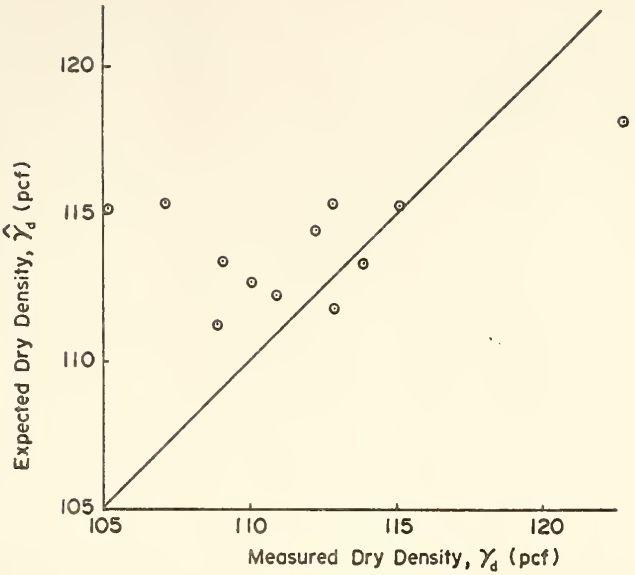



FIGURE 22 EXPECTED VS MEASURED DRY DENSITY: SHEEPSFOOT ROLLER, AS-COMPACTED

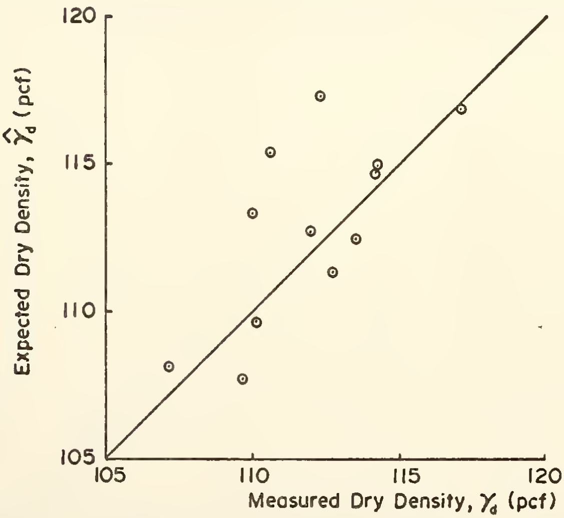



FIGURE 23 EXPECTED VS MEASURED DRY DENSITY: RUBBER-TIRED ROLLER, AS-COMPACTED

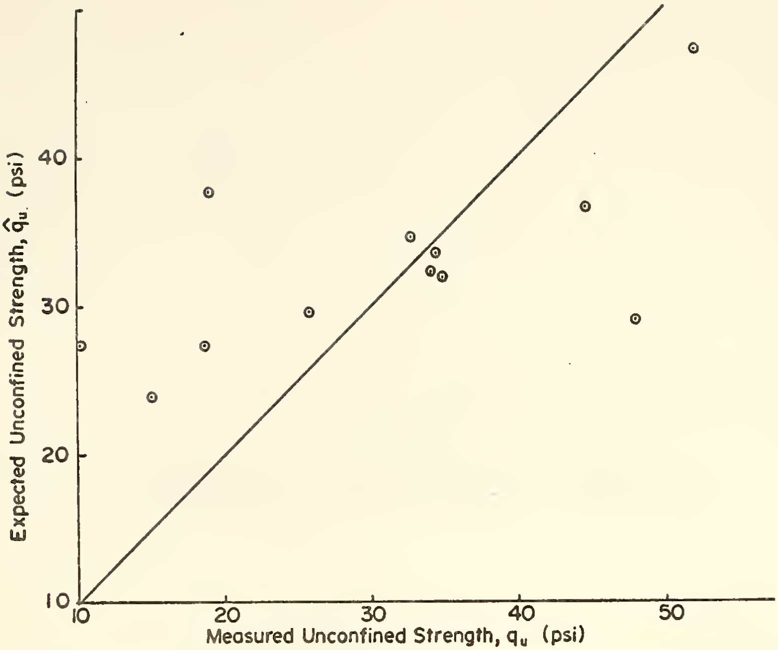



FIGURE 24 EXPECTED VS MEASURED UNCONFINED STRENGTH: SHEEPSFOOT ROLLER, AS-COMPACTED

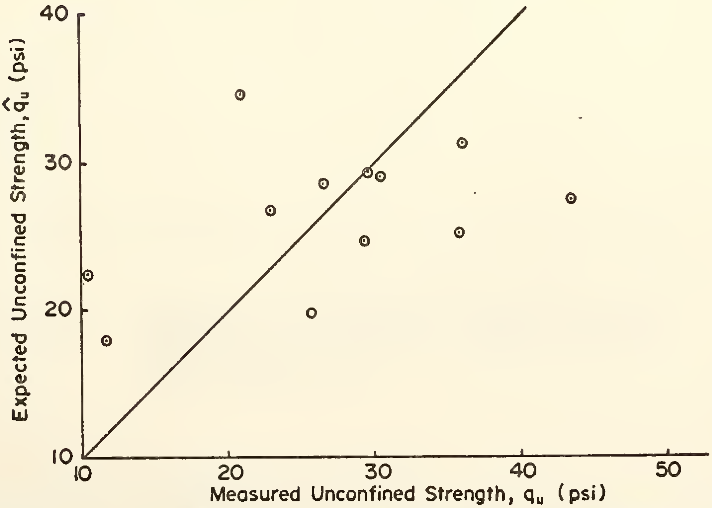



FIGURE 25 EXPECTED VS MEASURED UNCONFINED STRENGTH: RUBBER-TIRED ROLLER, AS-COMPACTED

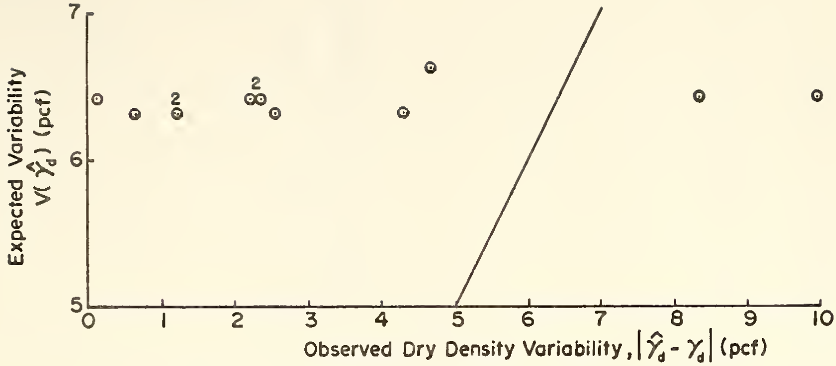



FIGURE 26 EXPECTED VS OBSERVED DRY DENSITY VARIABILITY: SHEEPSFOOT ROLLER, AS-COMPACTED

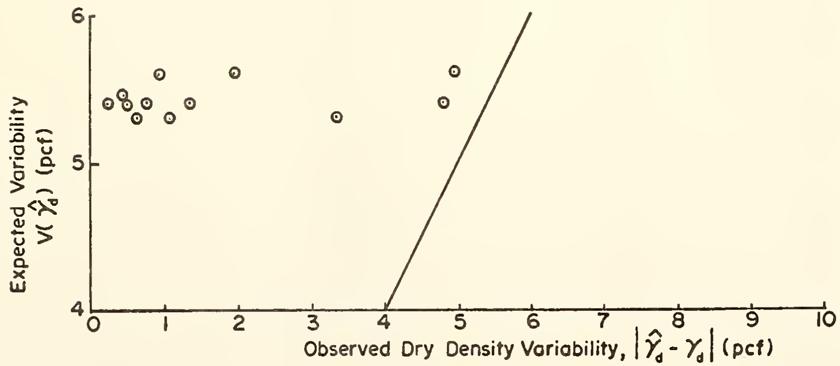



FIGURE 27 EXPECTED VS OBSERVED DRY DENSITY VARIABILITY: RUBBER-TIRED ROLLER, AS-COMPACTED

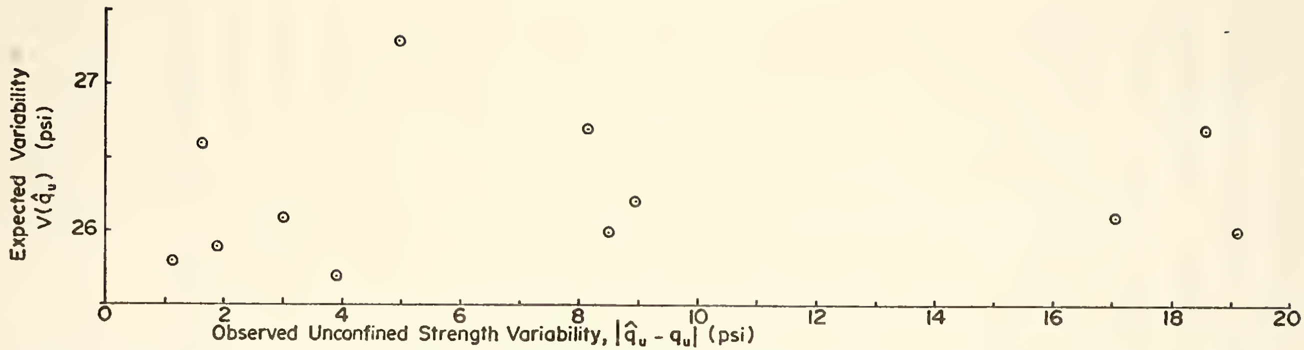



FIGURE 28 EXPECTED VS OBSERVED UNCONFINED STRENGTH VARIABILITY: SHEEPSFOOT ROLLER, AS-COMPACTED

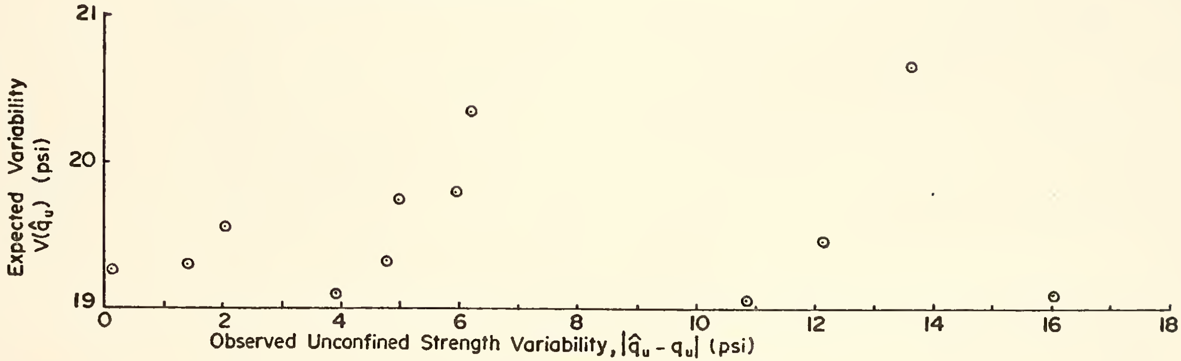



FIGURE 29 EXPECTED VS OBSERVED UNCONFINED STRENGTH VARIABILITY: RUBBER-TIRED ROLLER, AS-COMPACTED

Few conclusions can be drawn from Figures 22, 23, 24 and 25 without simultaneously reviewing the variability expected among the plotted points. These graphs show that a large difference may exist between an expected value of the parameter and the corresponding value observed during testing. This does not mean that the model poorly represents the relationship between the variables; it suggests that the condition of a compacted soil after construction is very heterogeneous. Therefore, any one measurement of either the dry density or strength parameter may be unrepresentative of the average value of the parameter and as such does not reflect the true quality of the compaction results. Figures 26, 27, 28 and 29 show that the observed variability of both the dry density and the strength is, in nearly all cases, lower than the expected variability. The observed variability for each data point is the vertical distance between the observed point and the 45° line in Figures 22, 23, 24 and 25 measured in the units of the ordinate. The expected variability calculation produces an upper bound value; it comes from a type of "worst-case" calculation. Because the "worst-case" does not always occur, one should expect the observed variability to be less than the expected variability; the difference can be large. Therefore, the magnitude of the scatter of points about the 45° equality line in Figures 22, 23, 24, 25 is a reasonable suggestion that this analysis is applicable as a prediction method.

# DISCUSSION OF RESULTS

## Field Compaction Results

### Dry Density and Strength Magnitude

Preceding analysis generated prediction models for dry density and for strength. Expressions for dry density are used to develop a quality assurance program based upon measurement of dry density rather than strength; this is shown in the later section entitled "Application of Results". In the expressions for strength, the dry density appears as a variable, as do water content and compactive energy. Because the dry density is already functionally related to water content and energy (i.e., not independent), it is believed more appropriate to not have all 3 variables in the prediction equation. Accordingly, the prediction equations for dry density were substituted into those for strength. This is statistically acceptable, and the ultimate justification rests upon a judgment as to whether the final equation is suitable. For purposes of this study, this procedure is considered suitable. Table 10 shows the prediction models before and after the substitutions were made. They are presented to facilitate the discussion of the sections which follow.

Table 10. Dry Density and Strength Regression Models.

| Roller Type and Soil Condition | Model Type | Regression Model |
|---|---|---|
| SFR-A.C. | Original | $\hat{\gamma}_d=-1.398w+133.43$ |
| | Original | $\hat{q}_u=-2.058w+.585E+.353\gamma d+16.99$ |
| | After Substitution | $\hat{q}_u=-2.551w+.585E+64.09$ |
| RTR-A.C. | Original | $\hat{\gamma}_d=-.0476w^2+.169E+122.46$ |
| | Original | $\hat{q}_u=-2.224w-.155E+.0734\gamma d+51.91$ |
| | After Substitution | $\hat{q}_u=-.00349w^2-2.224w-.143E+60.90$ |
| RTR-S. | Original | $\hat{\gamma}_d=1.529E-.0958wE+110.75$ |
| | Original | $\hat{q}_u=1.189w+.195E+.968\gamma d-118.37$ |
| | After Substitution | $\hat{q}_u=1.189w+1.675E-.0927wE-11.16$ |

Sheepsfoot Roller: As-Compacted

Figure 30 is an isometric presentation of the expected unconfined strength surface in the water content-compactive effort-strength space axes system. The surface is a plane and is represented by the appropriate regression model of Table 10. Of interest is that the strength decreases with increasing water content; a 1 percent water content difference produces a 2.55 psi change in the expected unconfined strength. Strength increases with additional compactive effort; a 1 pass compactive effort difference causes a change in the expected strength of 0.585 psi.

The regression results for these wet-side compaction data show that the dry density is not a function of the compactive effort. As shown on Figure 32, the dry density curve is nearly parallel to the zero air voids curve and due to the dry density-compactive effort independency, the model may represent the limit of compaction efficiency.

Figure 33 shows the relationship between the expected dry density and the expected unconfined strength. Even though the slopes of the two curves are unequal, prediction of the strength from the dry density may result if both the water content and compactive effort are known. To accomplish this for both the wet and dry side of the optimum water content however, a number of criteria must be met: 1) the optimum water contents must be established for each compactive effort level; 2) models representing the compaction curves for the dry side of the optimum must be found; 3) the peaks of the strength curves must be defined; 4) models representing the strength curves

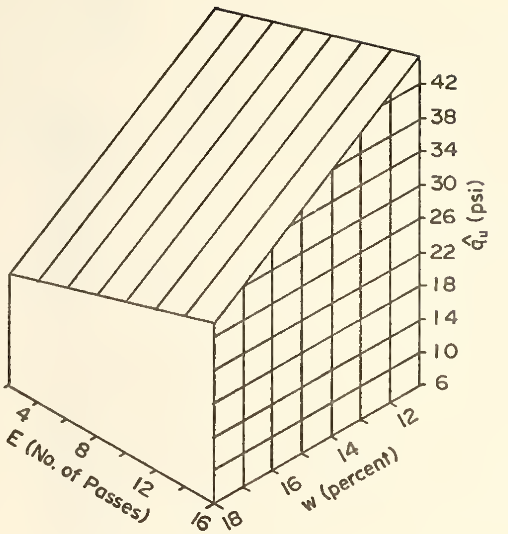



FIGURE 30 ISOMETRIC PRESENTATION OF THE EXPECTED UNCONFINED STRENGTH: SFR-A.C.

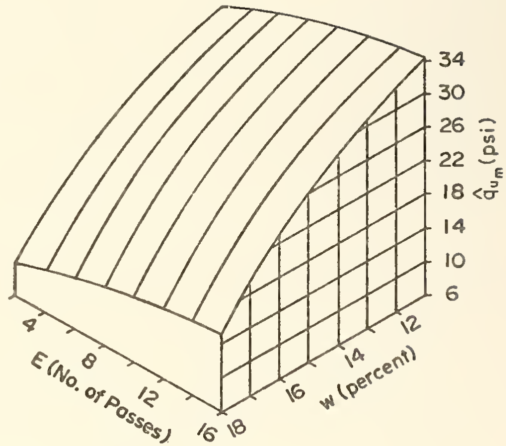



FIGURE 31 ISOMETRIC PRESENTATION OF THE MINIMUM EXPECTED UNCONFINED STRENGTH: SFR-A.C.

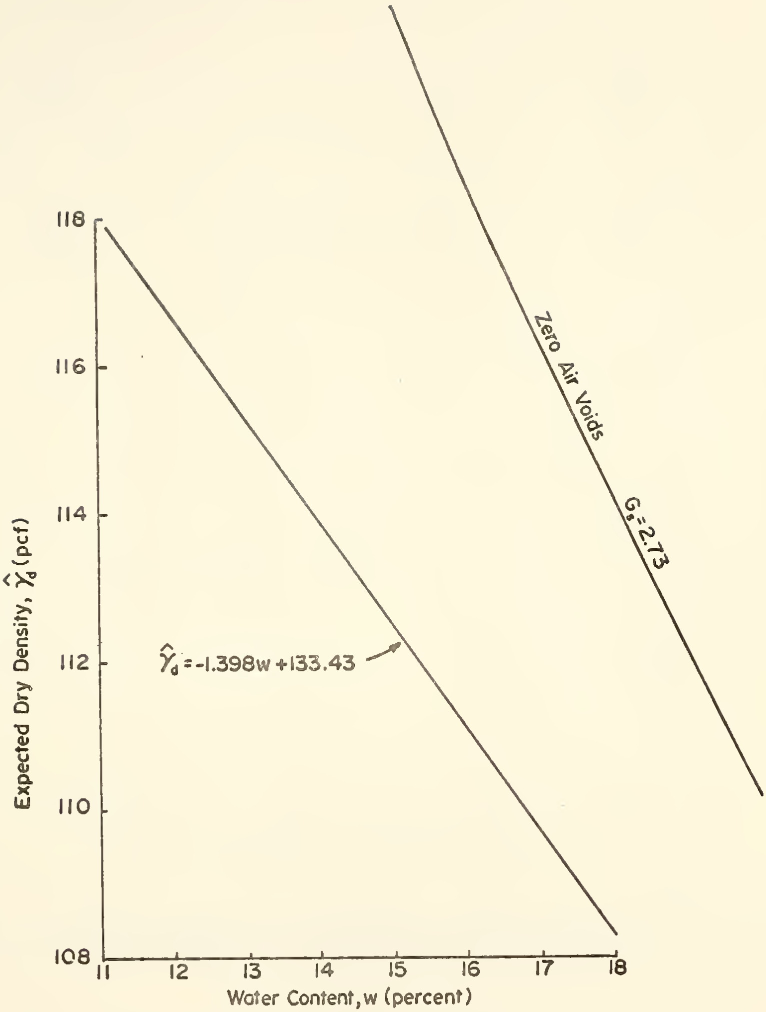



FIGURE 32 EXPECTED DRY DENSITY-WATER CONTENT RELATIONSHIP FOR SHEEPSFOOT ROLLER: AS-COMPACTED

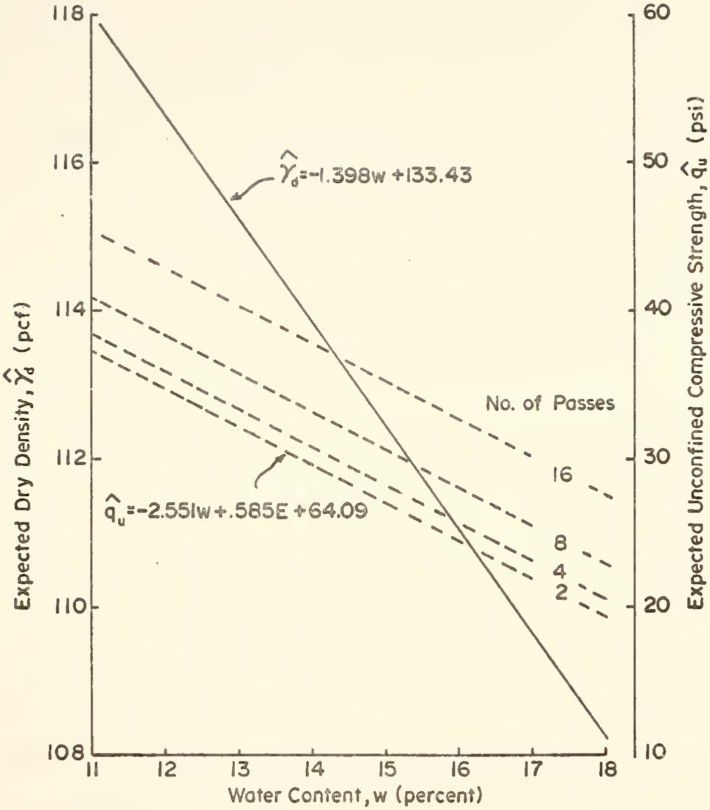



FIGURE 33 DRY DENSITY-STRENGTH RELATIONSHIP FOR SHEEPSFOOT ROLLER: AS-COMPACTED

for the dry side of these peaks must be defined; and 5) relationships between the density and strength models must exist and be defined. The first four criteria are relatively easy to accomplish. Reasons are given in the next section as to why they were not attained during this project. The fifth criterion may also be relatively simple to meet if the peaks in the strength curves coincide with the optimum water contents for each compactive effort level. Should the strength peaks and optimum water contents occur at different moisture levels, a complicated analysis will ensue, the results of which may be best presented as a graphical relationship such as Figure 33.

Rubber-Tired Roller: As-Compacted

The strength surface for the water content-compactive effort-strength space system is shown isometrically in Figure 34. As with the sheepsfoot roller, the strength decreases with increasing water content. Although the amount of the strength change is dependent upon the absolute magnitude of the water content, a 1 percent change in water content will result in an approximate 2.31 psi change in the expected strength. However, strength decreases as the compaction effort is increased; a 1 pass difference of the compactive effort results in a 0.143 psi change in the expected strength.

Dry density is dependent upon the compactive effort level; density increases with compactive effort as shown in Figure 36. The density curves become nearly parallel to the zero air voids curve for the higher water contents indicating that extrapolation of the density-energy relationship beyond the 16 pass range may be in error.

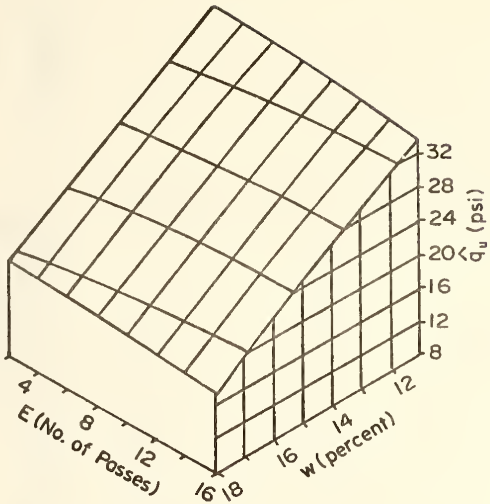



FIGURE 34 ISOMETRIC PRESENTATION OF THE EXPECTED UNCONFINED STRENGTH: RTR-A.C.

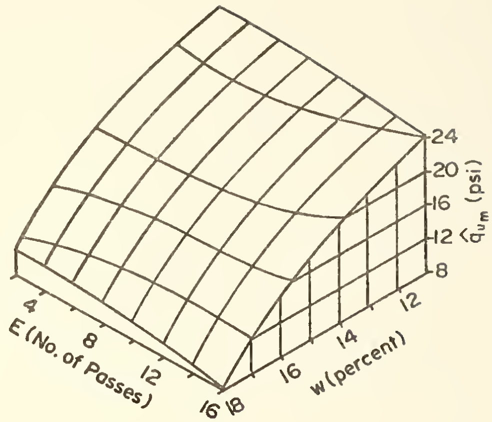



FIGURE 35 ISOMETRIC PRESENTATION OF THE MINIMUM EXPECTED UNCONFINED STRENGTH: RTR-A.C.

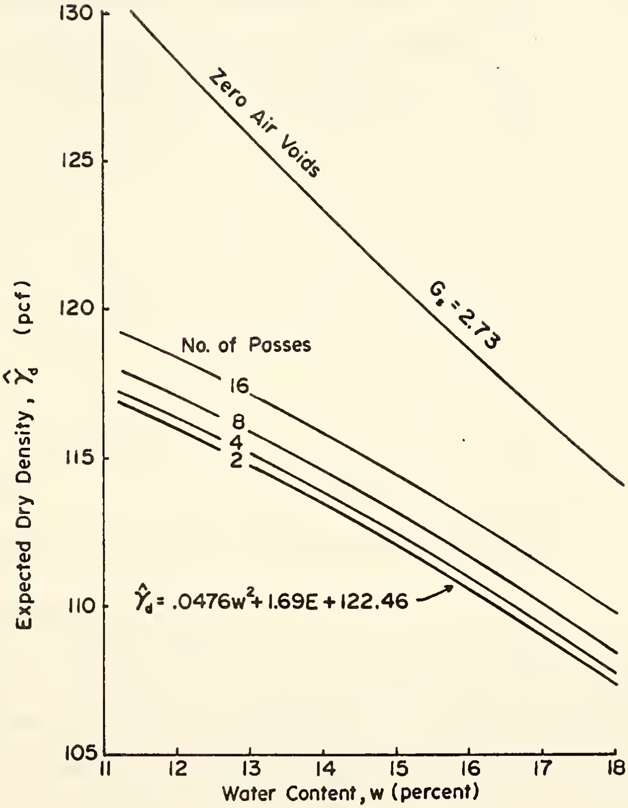



FIGURE 36 EXPECTED DRY DENSITY-WATER CONTENT RELATIONSHIP FOR RUBBER-TIRED ROLLER: AS-COMPACTED

If the limit of compaction efficiency has been nearly reached with 16 passes, a sizable increase of the compactive effort may cause only a small or neglegible increase in dry density for the wetter soils.

Figure 37 presents the relationship between the expected dry density and the expected unconfined strength. The slopes of the density curves are not only different from the slopes of the strength curves, but they are also changing with respect to the strength slopes. This complicates the strength-density prediction process although if the same criteria are met as described earlier, strength may be reasonably forecasted from knowing the water content, compactive effort and dry density of the soil mass. Of particular interest is that the density increases and the strength decreases with increasing compaction effort at a constant water content. What this indicates is that if an inspector requires additional compaction with intentions of increasing the density to some minimum specification values, that compaction may in fact reduce the strength; a form of over-compaction.

Rubber-Tired Roller: Soaked

Figure 38 depicts the expected strength surface in the water content-compactive effort-strength space grid system. Note that both the water content and compactive effort axes have been oriented in the opposite direction from those of the sheepsfoot and rubber-tired roller as-compacted graphs. The expected strength increases with increasing compactive effort; a 1 pass change in energy results in a 1.6 psi change in the strength. Unlike the as-compacted cases,

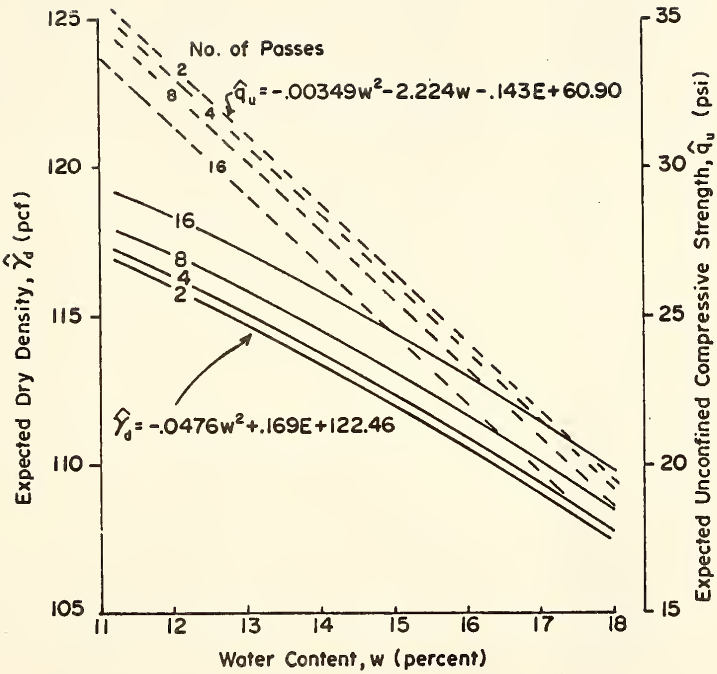



FIGURE 37 DRY DENSITY-STRENGTH RELATIONSHIP FOR THE RUBBER-TIRED ROLLER: AS-COMPACTED

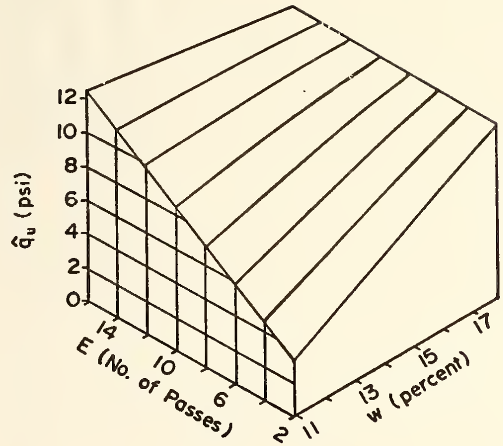



FIGURE 38 ISOMETRIC PRESENTATION OF THE EXPECTED UNCONFINED STRENGTH: RTR-S.

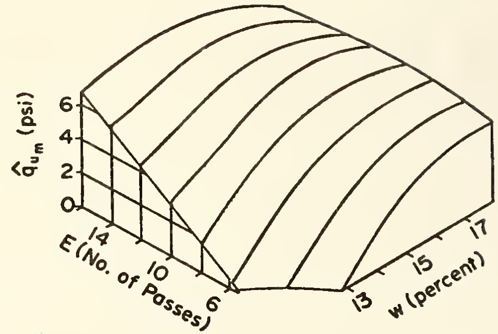



FIGURE 39 ISOMETRIC PRESENTATION OF THE MINIMUM EXPECTED UNCONFINED STRENGTH: RTR-S.

the strength increases with increasing water content -1.1 psi for every 1 percent difference in water content. A plausible explanation for this trend is that the samples compacted closer to a saturated condition may have swelled less (in percent of total volume) than those samples compacted at a lower water content; the inherent assumption is that the greater the swell the weaker the soil condition.

The density-water content relationship is shown in Figure 40. Of interest in this graph is the decreasing slope trend for decreasing compactive effort in the lower water contents. This trend suggests that at high compactive efforts, water content differences are more influential on the density magnitude than water content differences at lower compactive efforts. Again, this would suggest the swelling influence increases with increasing compaction energy.

The as-compacted dry density-water content relationship should be similar for the samples tested in the soaked condition and for those tested as-compacted. This is necessarily so because all of the samples were compacted and sampled in a like manner, the density and water content measurements for both sample groups were made prior to laboratory testing (including the soaking phase in the case of the samples saturated before testing), and the samples were separated in the two groups by a random process. By comparing Figures 36 and 40 and reviewing the density equations given in Table 8 or 10, it can be seen that a large discrepancy exists between the two relationships. Two causes are believed to have created the difference. The first possible cause for the discrepancy is that substantially fewer samples were tested in the soaked condition than were tested as-compacted.

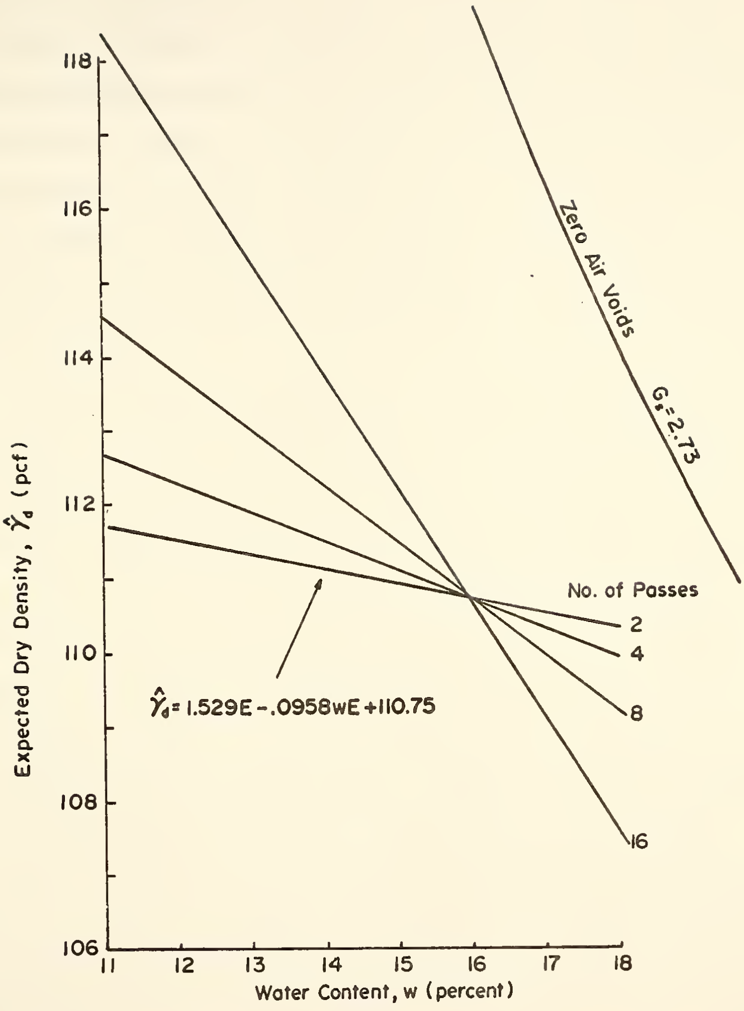



FIGURE 40 EXPECTED DRY DENSITY-WATER CONTENT RELATIONSHIP FOR RUBBER-TIRED ROLLER: SOAKED

Samples (regarding all the as-compacted samples as one sample and the soaked samples as another sample) containing unequal data quantities may represent the desired relationship in different ways. However, the true population relationship between the dry density, compactive effort and water content is more likely to be represented from a sample containing the greatest quantity of data. For this reason, it is believed that the as-compacted water content-dry density relationship is perferable as a predictor of the true relationship of the test lift.

The second possible cause for the difference in compaction curves is that the entire tested sample was used for water content determination for the as-compacted samples whereas only the end trimmings were used to determine the molding water content for the samples tested in the soaked condition. As mentioned in the "Analysis" section, zones of differing water contents existed in many samples. Trimming of the ends through the deeper sections was impossible. Thus, the dryer portions had to be removed to obtain an adequate end surface and for the soaked samples, these end trimmings were used for the determination of the water content used in the analysis. Due to this difficulty, it is believed that the water content variations in the end trimmings may have prevented an accurate measurement of the sample water content. Although water contents were determined after shearing for the soaked samples, no measurements of the volume of water retained by the sample during the soaking process were taken. This prevented back-calculations of the before-soaking water contents. Because of the uncertainty of correct water content

evaluations, further testing is needed to verify the results presented here.

Figure 41 shows the association between the expected dry density and the expected unconfined strength. If, in fact, the relationships for the soaked dry density and strength are as shown, the prediction process should be relatively simple, much like the process used for the as-compacted samples of the sheepsfoot roller. Of importance is the sign difference of the slopes for the two curve types in the lower energy levels. The dry density is decreasing with additional water content while the strength is increasing. This suggests that if peaks exist for both curves, the water content at which strength is a maximum is significantly greater than the water content at which the density curve peaks for the lower energy levels. For greater compactive efforts, strength and density decrease simultaneously as the molding water content is increased.

As expressed above in the discussion of the dry density model, it is believed that the molding water contents may not have been satisfactorily obtained. This results in density and strength models being defined that may not be representative of the true relationships that existed in the test pad. Therefore, verification of these models is desired before they are extensively used.

Sheepsfoot Roller: Soaked

Only twelve samples compacted by the sheepsfoot roller were tested in the soaked condition (see p. 45 for reason why sample number was so low). The scatter-plot of the dry density versus the

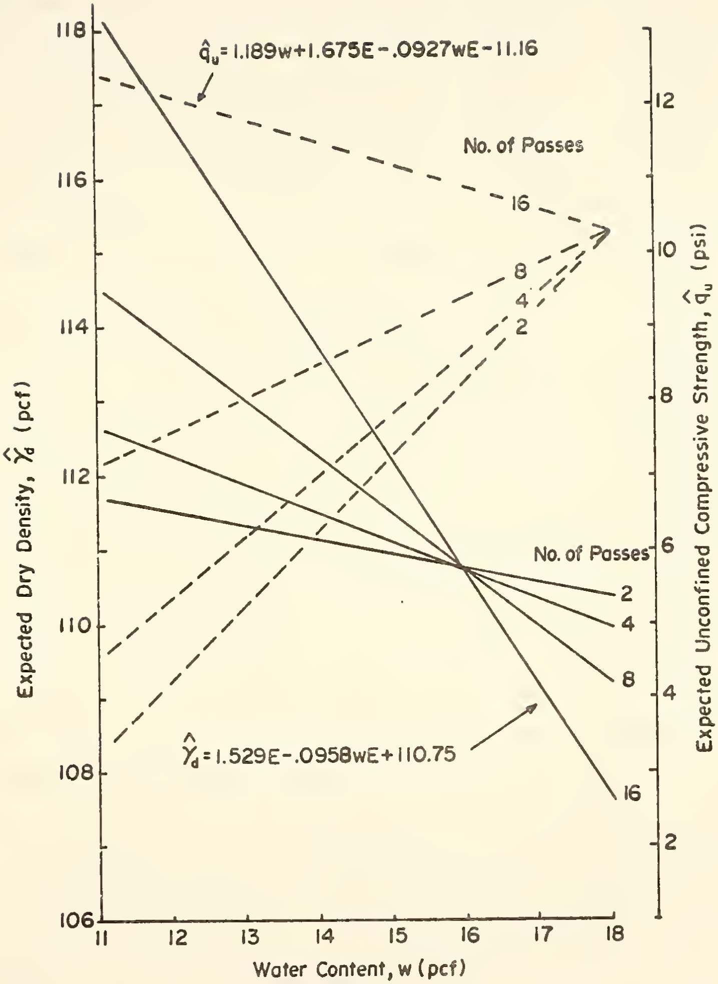



FIGURE 41 DRY DENSITY-STRENGTH RELATIONSHIP FOR THE RUBBER-TIRED ROLLER: SOAKED

water content showed a large amount of scatter in the data. It is believed that much of this scatter was caused by an inappropriate water content being correlated with a given density due to the use of end trimmings for the water content determinations. As a result of the large scatter and small number of data points, no significant prediction model could be defined for the compaction curve. Since the analyses depend upon the density-water content-compactive effort relationship, nothing further was attempted.

## Variability in Dry Density and Strength

Figures 31, 35 and 39 show the minimum expected unconfined strength surface in the water content-compactive effort-strength space system for the as-compacted soil conditions of the sheepsfoot and rubber-tired rollers and the soaked rubber-tired roller, respectively. The minimum unconfined strength is defined as the expected strength minus the expected variability in the strength. In each case, the surface is significantly different from the expected strength surface. Similar figures for the dry density surfaces would show comparable differences between the expected and minimum expected surfaces. Illustrated in these graphs is the notion that the variability in compaction results is significant and must be provided for in the project specifications to ensure the design engineer a minimum strength compatible with the project needs.

As discussed in the "Project Purpose" section, variation in the results of a compacted soil are expected due to varying soil conditions, compaction processes and testing procedures.

Some variation in the sampling and handling process at the test pad was inevitable because of the number of personnel required to collect the samples. However, once at the Purdue soil's laboratory, all samples were treated in a like manner. No quantitative measurements of the sampling and testing variability were made.

Part of the observed data scatter is thought to have originated from variations in the compaction process. Although the gross weights of the two rollers were kept constant, varying speeds of operation and dissimilar total compactive effort distributions existed. The sheepsfoot roller had two operators, each of which ran the machine at a consistent velocity but different from one another. This was not a problem for the rubber-tired roller as one man operated the machine throughout the project. Therefore, the data scatter caused by varying operational speeds are believed to be larger for the sheepsfoot roller than for the rubber-tired roller.

Total compactive effort produced by one pass of each roller varied from location to location between the test lifts and between each successive effort level of each test lift. On every pass of the sheepsfoot roller, only a portion of the soil was directly under a foot of the roller. Increasing the number of passes increased the opportunity that all locations of the soil had been compacted under a foot. However, increasing the passes also increased the probability that any one location could be compacted under a foot two or more times. Therefore, there existed some distribution of total compactive effort in the test lift for each energy level that contributed to the variability found in the data results. This variation is regarded as

inherent with normal construction practices; therefore, the associated variability cannot be avoided. A different total compactive effort distribution existed for the soil rolled by the rubber-tired roller. On occasion during the compaction process and usually while rolling the wetter test-lift sections, soil squeezed between the tires and prevented further roller movement. The light dozer used for the disking operation had to be employed as a push-cat to move the roller and complete the pass. This caused at least two variations in the total compactive effort distribution. 1) The roller was stationary for varying amounts of time over various portions of different test lifts and 2) the speed of the roller was not kept constant for the entire compaction process. Again, no quantitative measurements were taken for these variabilities. It is believed that these process variations may have caused a significant increase in the data scatter.

The third source of compaction result variability is dissimilarity of soil conditions at the time of compaction. Small variations in soil type and condition within the borrow area were expected although no measurements were taken. These variations are considered inherent for normal construction practices. Temperature changes during compaction occurred causing some variation in response of the soil to compaction. Since compaction for any project will occur during substantial temperature changes, this variability source is again expected and along with the borrow material variability, should be accounted for in the design specification and quality assurance program.

Already mentioned in the "Analysis" section was that the average water content variation within any section of the test lifts was 3.3 and 3.7 percent for the sheepsfoot and rubber-tired rollers, respectively. Stratified zones of differing water contents were observable in many samples, indicating that the soil was not of uniform consistency at the time of compaction or at the time of testing. Figure 42 illustrates the layering effect found in some samples. Although some variability of water content within a sample is expected, the magnitude of the variation found in this project was considered the primary cause of the large data scatter.

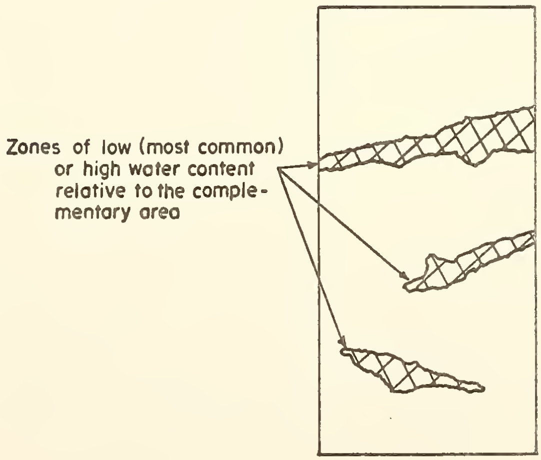



FIGURE 42 LAYERING EFFECTS OF WATER CONTENT VARIATIONS FOUND IN SOME SAMPLES

By reducing the variability in the data, the probability of the regression model representing the true relationship between the

variables would be increased and the expected variability would be reduced. Figure 43 shows the effect that reducing the water content uncertainty has upon the expected strength variability. As shown, for any given water content and compactive effort, a higher degree of water content homogeneity results in a significantly reduced strength variation. Since the design engineer is interested in accurately forecasting the expected strength, compactive specifications should include provisions that reduce the variability in the soil and compaction process.

## Verification of Regression Models

The expected variability graphed in Figures 26, 27, 28 and 29 is the half range of the 95 percent confidence interval associated with individual data points (i.e., based on the standard deviation of the data). Because of the large magnitude of this variability, design application practicality is severely limited. Little use can be made of knowing that a particular soil condition and compaction process will yield on a point-to-point basis, a resultant expected strength of 40 psi and an expected variability of plus or minus 30 psi. If, however, a number of samples are used to represent a fill lift or portion thereof, the average value of these samples may be compared with expected strength plus or minus the variability derived from the standard error of the mean; this is a much smaller and thus, more useful variability measurement. Table 11 lists the results for the same 24 points as in Figures 26, 27, 28 and 29 using the samples' average strength for the observed strength and the standard

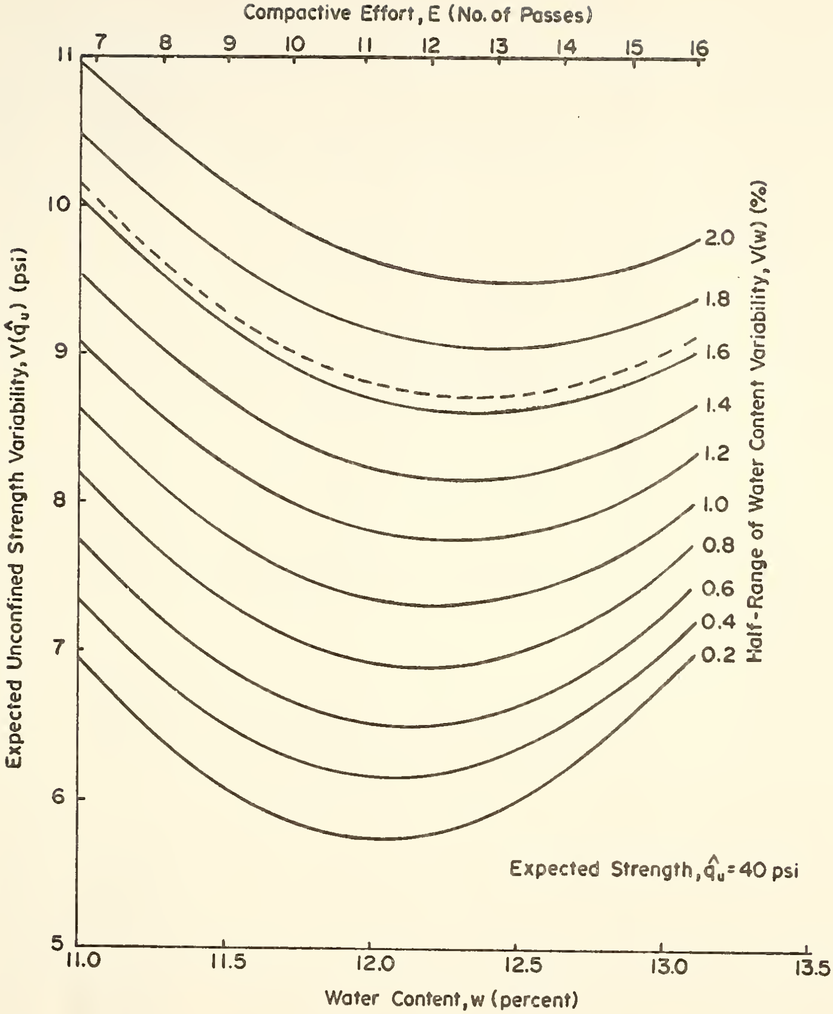



FIGURE 43 EFFECT OF WATER CONTENT VARIABILITY ON UNCONFINED STRENGTH VARIABILITY, SHEEPSFOOT ROLLER, A.C. - 40 PSI EXPECTED STRENGTH

Table 11. Verification of Regression Results Using the "Standard Error of the Mean" Approach.

| Roller Type | Number of Passes | Number of Samples | Expected Strength | Observed Strength | Expected Variability | Observed Variability |
|---|---|---|---|---|---|---|
| SFR-A.C. | 2 | 5 | 28.0 psi | 25.3 psi | 6.7 psi | 2.7 psi |
| | 4 | 1 | 32.9 | 33.9 | 7.6 | 1.0 |
| | 8 | 2 | 31.7 | 30.1 | 5.7 | 1.6 |
| | 16 | 4 | 40.3 | 37.1 | 9.2 | 3.2 |
| RTR-A.C. | 2 | 4 | 25.2 | 23.7 | 5.8 | 1.5 |
| | 4 | 2 | 31.5 | 32.1 | 7.6 | 0.6 |
| | 8 | 4 | 23.8 | 26.8 | 5.8 | 3.0 |
| | 16 | 2 | 26.8 | 27.8 | 6.5 | 1.0 |

error of the mean for the expected variability. In all cases, the observed strength lies within the range defined by the expected strength plus or minus the expected variability. In comparison to Figures 26, 27, 28 and 29, the expected variabilities in Table 11 are small enough so that application of the expected strength and associated range for design and quality control purposes is practical.

Therefore, to ensure an accurate measure of the field strength or density, the average of a number of sample values should be used to represent the parameter in question. Regardless of whether the inspector chooses to sample from the entire lift on a statistical basis or from only that portion of the fill he believes is questionable, more than one or two samples must be taken to accurately determine the true quality of the work and prevent an inaccurate engineering judgment.

Laboratory and Field Correlation*

Figures 44 and 45 show the dry density regression models for the laboratory as-compacted soil and the field as-compacted soil superimposed. Note the similarity of the slopes for the wet-of-optimum compaction region; the likeness suggests that the two curves may be similar in shape. However, a complete correlation between the dry density models for either roller type and the corresponding

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*The laboratory as-compacted models for dry density and unconfined compressive strength were determined by Essigmann (1976) and the regression models for the laboratory compacted soil, tested in the soaked condition, were developed by Scott (1977). Listed in Table 4 are the properties of both of laboratory and field soils.

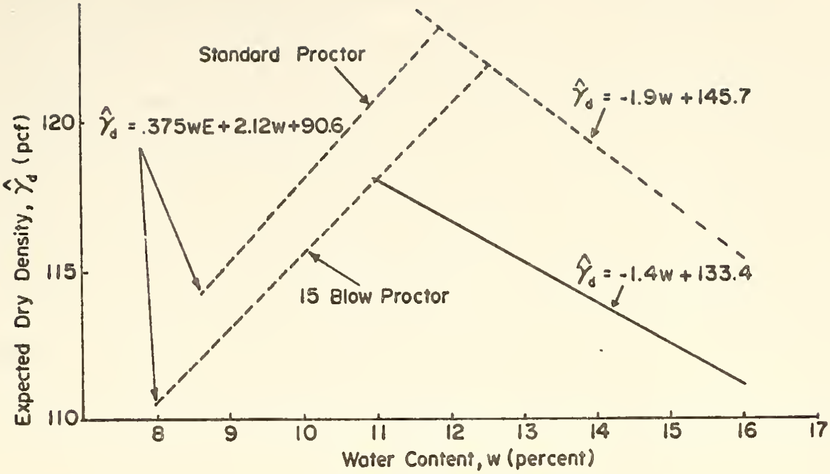


FIGURE 44 LABORATORY TO FIELD DENSITY CORRELATION: SHEEPSFOOT ROLLER, A.C.

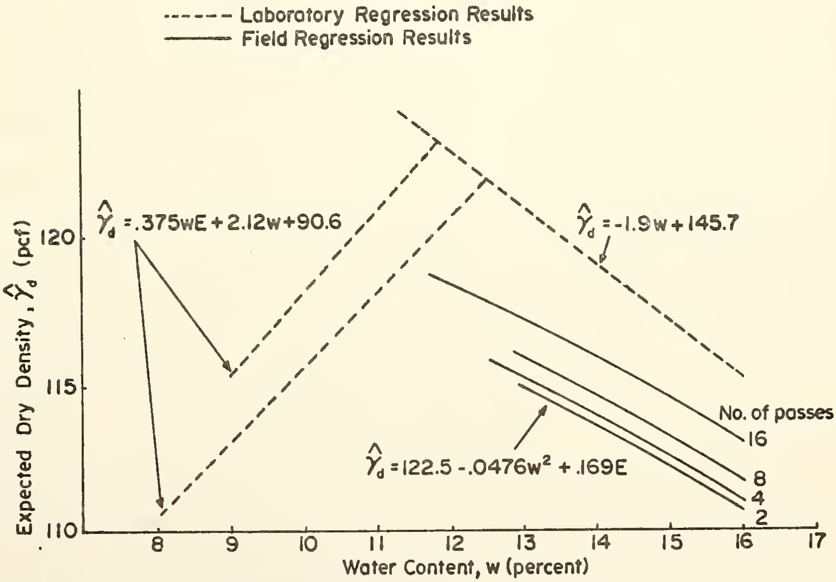


FIGURE 45 LABORATORY TO FIELD DENSITY CORRELATION: AND RUBBER-TIRED ROLLER, A.C.

laboratory models cannot be determined because the field optimum water content was not defined. Figure 46 presents the superimposed dry density regression models for the laboratory compacted soil and the rubber-tired roller compacted soil, both of which were tested in the soaked condition. Although the slopes are quite different for the lower field energy regions, they become nearly parallel for the higher regions. Again, without the establishment of the field optimum water content and thus the relative orientation of the laboratory and field curves, a complete orientation is not possible.

The field and laboratory strength regression models are presented in Figures 47 and 48 for the as-compacted soil condition. Figure 49 shows the soaked laboratory and field strength models superimposed. Again, verification of the field model is needed before this relationship is used extensively. Complete correlations between the laboratory and field compacted soils for both soil conditions are not possible without the establishment of the water contents at which the field curves peak.

Although the slopes of the field and laboratory dry density and strength curves are similar, they are not identical. For this reason, a mathematical equation or a set of equations that correlate the field results to laboratory tests will be complicated at best. However, the simple procedure of superimposing equational models as has been done in Figure 44 through Figure 49 may prove to be a viable and more useful method of predicting the field results from laboratory tests. A test pad whose results define the water contents of the field curve peaks should enable such a graphical procedure to be developed.

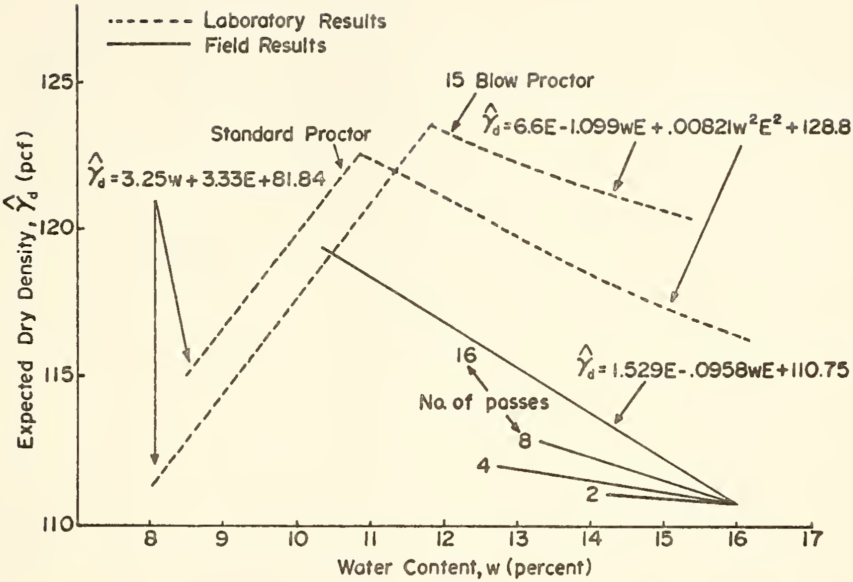


FIGURE 46 LABORATORY TO FIELD DENSITY CORRELATION: RUBBER-TIRED ROLLER, SOAKED

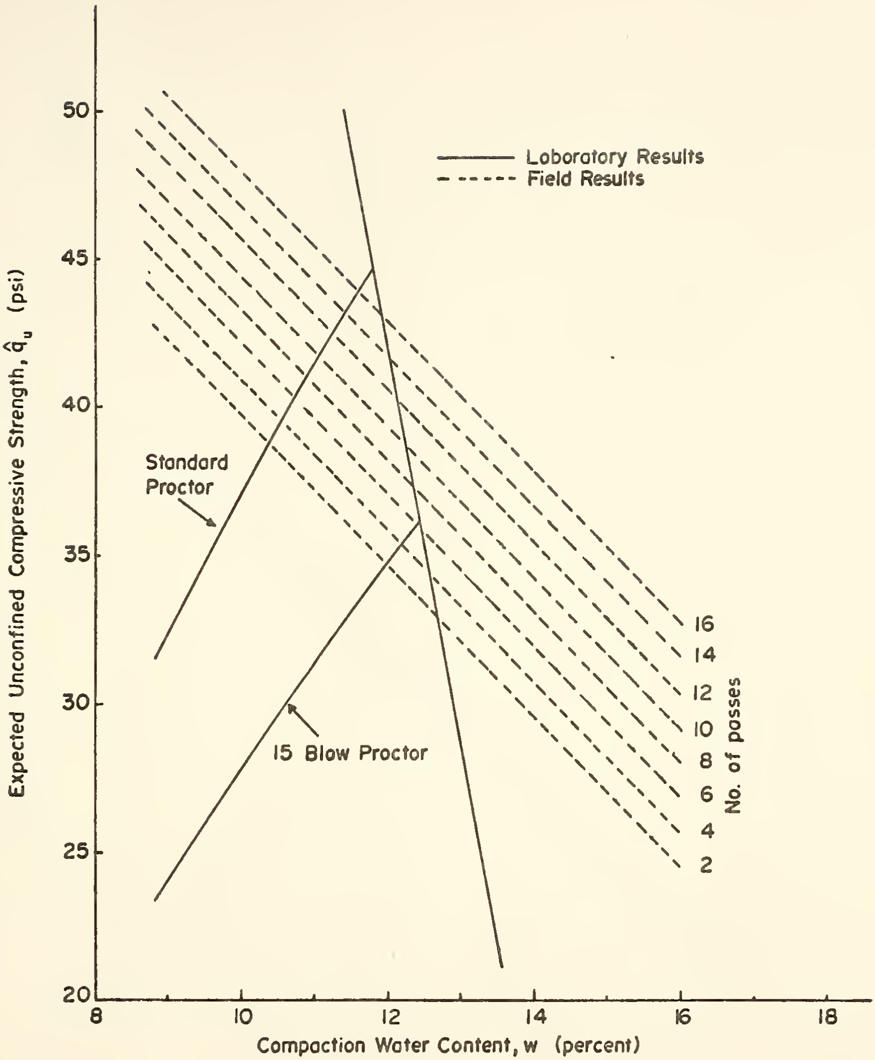


FIGURE 47 LABORATORY TO FIELD STRENGTH CORRELATION: SHEEPSFOOT ROLLER, AS-COMPACTED

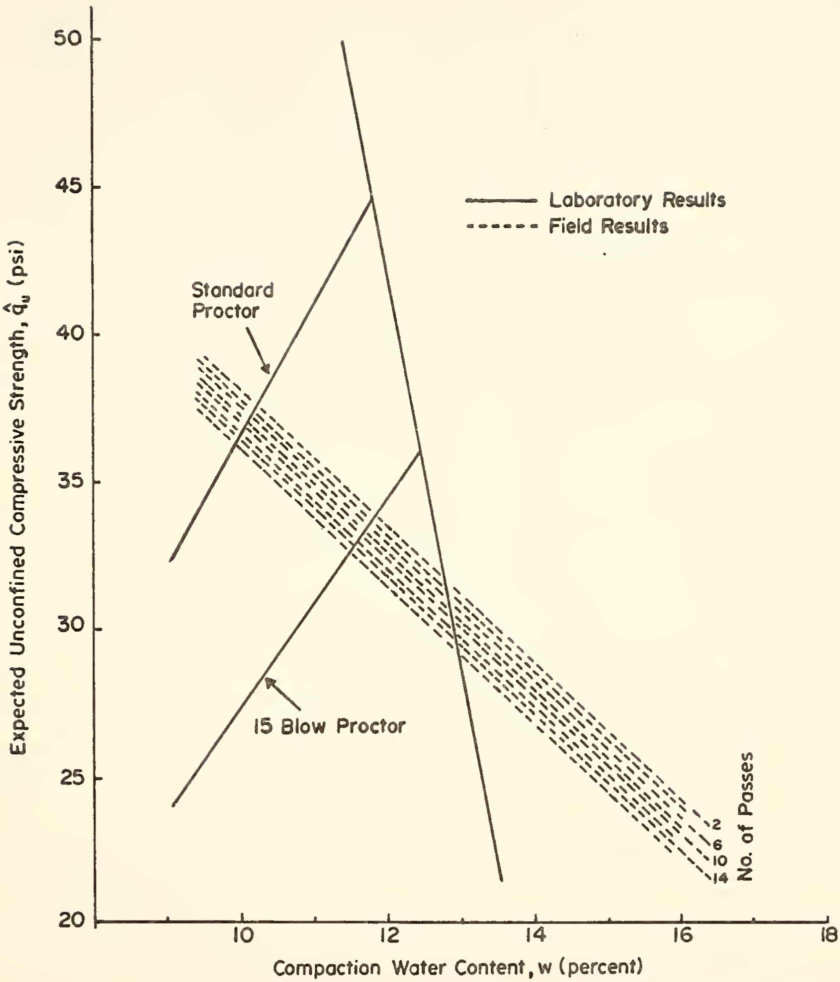


FIGURE 48 LABORATORY TO FIELD STRENGTH CORRELATION: RUBBER-TIRED ROLLER, AS COMPACTED

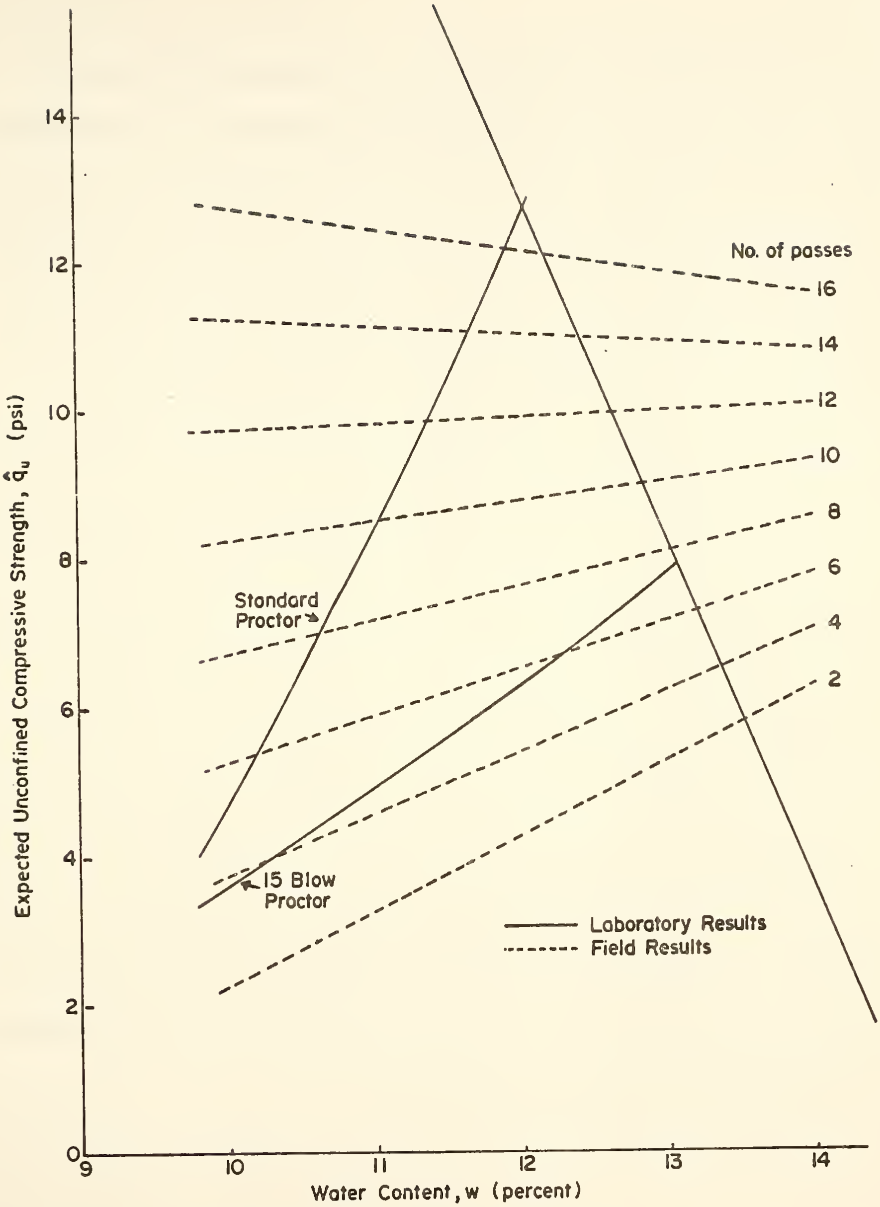


FIGURE 49 LABORATORY TO FIELD STRENGTH CORRELATION: RUBBER-TIRED ROLLER, SOAKED

The need for establishing the laboratory-field correlation is imperative to the economical implementation of this research. It is hoped that a number of test pads and similar research programs will follow to derive a suite of density and strength curves for various soil types and compaction processes. Without the corresponding research similar to that of Essigmann (1976) and Scott (1977) and the proper correlstion between the laboratory and field curves, the design engineer must require a test pad for each soil type to determine where the soil from that borrow area matches the suite of curves and, therefore, how to regulate the specifications to obtain the desired end-results. If, however, the correlation between the laboratory and field compacted soils can be made, the design engineer can simply take bag samples from the borrow area, run compaction and strength tests and from the results, determine where the soil matches the suite of curves. Therefore, with the proper laboratory-to-field correlation relationships, the design engineer can extrapolate the results of a relatively small number of test pads and corresponding laboratory studies to an infinite number of project soils without having to resort to extensive and expensive field sampling and testing programs.

# APPLICATION OF RESULTS

## Design Engineering

To insure proper performance of a compacted fill or embankment, the design engineer must rely upon the soil strength characteristics developed as a result of the compaction process. Slope stability and bearing capacity are two considerations of a fill design that depend directly upon the soil strength. Presently, most compaction specifications directly or indirectly use percent compaction as the controlling parameter of strength development. This study presented evidence that the percent of maximum density may not be the only factor that determines the resultant field strength. Therefore, a specification based directly upon the strength response to soil conditions and compaction processes is of greater use to the design engineer to assure stability of a low embankment slope or adequate bearing values of compacted subgrade.

The following discussion uses examples and accompanying commentary to illustrate the procedure for developing a strength-based compaction specification for a silty clay soil. All figures and data presented herein were developed by the analysis previously discussed. Figure 43 has been renumbered as Figure 50 and presented here for discussion convenience.

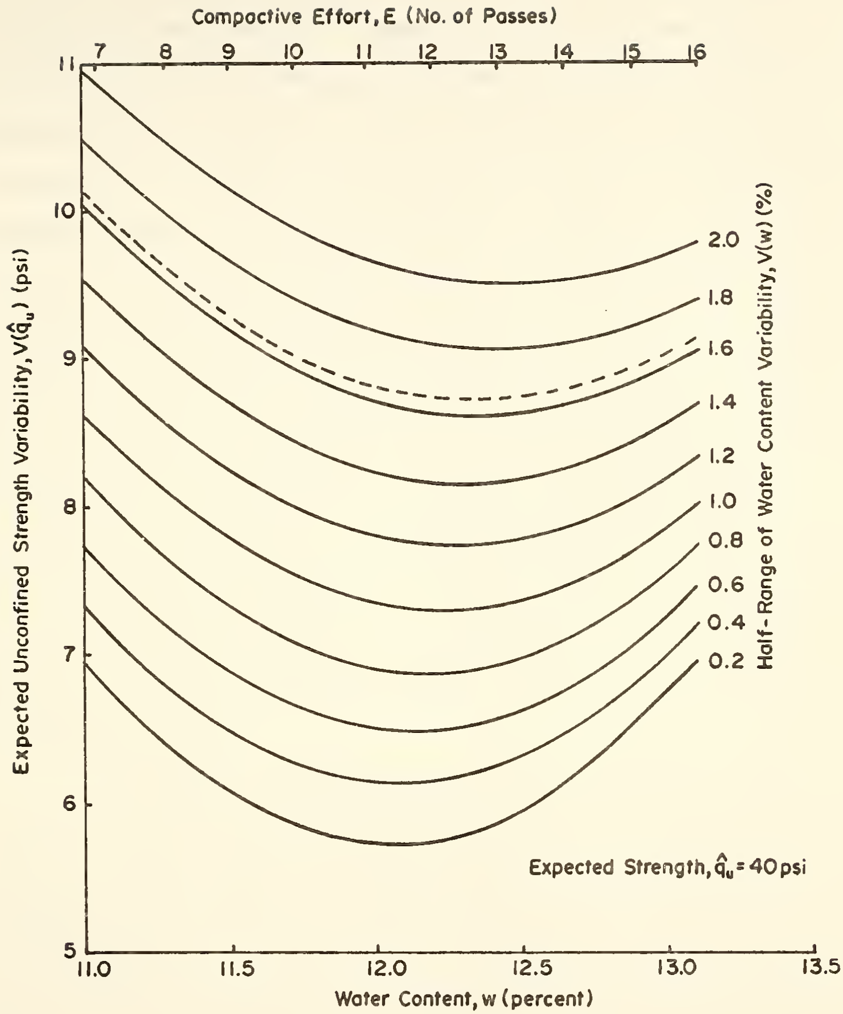


FIGURE 50 DESIGN GRAPH FOR 40 PSI EXPECTED STRENGTH: SHEEPSFOOT ROLLER, AS-COMPACTED

As an example, let the design dictate that a minimum required as-compacted soil strength necessary for satisfactory embankment performance be 32 psi. This value represents the minimum expected unconfined shear strength derived from subtracting the expected variability in strength from the expected strength ($V(\hat{q}_u)_m = \hat{q}_u - V(\hat{q}_u)$). An infinite number of $\hat{q}_u - V(\hat{q}_u)$ combinations exist that result in a 32 psi soil strength condition: 44-12, 40-8, 37.5-35.5, 34-2, etc. However, both the maximum expected strength and the minimum expected variability are limited by the roller type and the homogeneity of both the compaction process and the soil condition; combinations such as 90-58 and 32-0 are unrealistic. Should the engineer desire a uniform post-compaction soil condition, a compaction process resulting in the lowest possible expected variability should be specified. If a higher expected strength is more important than strength uniformity, a different compaction process may be better suited to produce such results.

Let us assume a process yielding a 40 psi expected strength and an 8 psi expected strength variability has been chosen to yield the minimum 32 psi criterion. Compaction by a sheepsfoot roller is expected. Thus, Figure 50 is used to determine the constraints of the specification. As shown, many combinations of compactive effort, water content and variability in the water content (E, w, V(w)) will produce a strength variability no greater than 8 psi. Reading from the ordinate axis to the right, the following soil conditions and compactive effort combinations are usuable: (7, 11.05, 0.77), (8, 11.28, 0.97), (9, 11.50, 1.10), (10, 7.3, 1.20), (11, 11.97, 1.30),

(12, 12.20, 1.33), (13, 12.42, 1.32), (14, 12.65, 1.27), (15, 12.88, 1.17) and (16, 13.1, 1.00). Note that the first number within the bracket represents the number of roller passes. The second number is the water content at which compaction should take place and the third number is the half-range of the water content variability in percent. Notice that the expected strength variability will not be exceeded if the water content variabilities are less than those shown above. If any of these combinations are specified, the design engineer can be confident that the compacted soil strength attained will meet the requirements of the $(q_u)_m$=32 psi design.

Figures 51, 52 and 53 represent similar graphs for the sheepsfoot roller corresponding to expected strengths of 25, 30 and 35 psi, respectively. Similar graphs for the as-compacted and soaked rubber-tired roller soil conditions are shown in Figures 54 through 57 and Figures 58 through 61, respectively.

## Quality Assurance Testing

The graphs described above are not convenient to the quality assurance engineer who is primarily interested in the results of the compaction process. Therefore, Table 12 is presented which represents a portion of the computer output of the program described in Appendix B. To use the tabulated form the engineer needs to measure the water content and the variability in the water content of the embankment section under consideration, and he needs to count the number of passes of the roller on the project. From this, the engineer is able to predict the expected strength magnitude and variability.

Table 12. Strength Variability Computer Program Output Representation.

| | | For Sheepsfoot Roller, A.C.: V(w) = 1.65 Percent | | | | | |
|---|---|---|---|---|---|---|---|
| Water Content (Percent) | Compactive Effort (Pass #) | Expected Dry Density (PCF) | Expected Strength (PSI) | Expected Dry Density Variability (PCF) | Expected Strength Variability (PSI) | Expected Min. Dry Density (PCF) | Expected Min. Strength (PSI) |
| 11.00 | 9.00 | 118.5 | 41.30 | 2.28 | 10.00 | 115.78 | 31.30 |
| 12.00 | 9.00 | 116.65 | 38.75 | 1.91 | 8.64 | 114.74 | 30.11 |
| 13.00 | 9.00 | 115.26 | 36.20 | 1.57 | 7.38 | 113.69 | 28.82 |
| 14.00 | 9.00 | 113.86 | 33.65 | 1.25 | 6.30 | 112.60 | 27.35 |
| 15.00 | 9.00 | 112.46 | 31.10 | 1.21 | 6.19 | 111.25 | 24.19 |
| 16.00 | 9.00 | 111.06 | 28.54 | 1.51 | 7.36 | 109.55 | 21.18 |
| 17.00 | 9.00 | 109.66 | 25.99 | 1.86 | 8.70 | 107.81 | 17.30 |
| 18.00 | 9.00 | 108.27 | 23.44 | 2.22 | 10.11 | 106.05 | 13.33 |
| 11.00 | 10.00 | 118.05 | 41.88 | 2.28 | 10.03 | 115.78 | 31.86 |
| 12.00 | 10.00 | 116.65 | 39.33 | 1.91 | 8.69 | 114.74 | 30.64 |
| 13.00 | 10.00 | 115.26 | 36.78 | 1.57 | 7.48 | 113.69 | 29.31 |
| 14.00 | 10.00 | 113.86 | 34.23 | 1.25 | 6.46 | 112.60 | 27.78 |

Again, the variabilities shown in this table are based upon the standard error of the mean; therefore, a number of samples (say 5 to 7) must be taken from the area of interest; the appropriate sample number depends upon the confidence the engineer wishes to have in the test results. As more samples are taken, the more confidence the engineer may have.

For each sample taken, measure the water content and density. From the group of samples, calculate the average water content and dry density and calculate the expected variabilities in the water content and dry density by calculating the half range for each parameter. Each section of the table is based upon a corresponding water content variability for the particular roller type and soil condition (as-compacted or soaked). Table 12 is based upon the water content variability found at the Anderson test pad which was 3.3/2 = 1.65 for the sheepsfoot roller. Proceed into the appropriate table section using the half range of the water content as the variability measurement. Use the calculated average water content to locate the proper position in the left-hand column (1), and the number of roller passes to locate the proper position in the next column (2). Starting with the third column, the rest of the row may be read as follows:

| Column Number | Explanation |
|---|---|
| 3 | Expected Dry Density, $\gamma d$ in PCF |
| 4 | Expected Unconfined Shear Strength $\hat{q}_u$ in psi |
| 5 | Expected Dry Density Variability, $V(\hat{\gamma} d)$ in PCF |

| | |
|---|---|
| 6 | Expected Shear Strength Variability, $V(\hat{q}_u)$ in psi |
| 7 | Minimum Expected Dry Density, $(\hat{\gamma}_d)_m$ in PCF |
| 8 | Minimum Expected Unconfined Strength, $(\hat{q}_u)_m$ in psi. |

Of particular interest are columns 4, 5 and 8. Columns 4 and 8 show what the average and minimum unconfined shear strength will be. Column 5 can be used as a check for the compaction process. If the calculated dry density variability is greater than the value shown in column 5, the values in columns 6 and 8 may be in error. Just as an increase in water content will increase the expected strength variability, so will a dry density variability increase the strength variability. Because the calculation of this density variability provides a convenient check for the application of this study, the strength was defined as a function of the density rather than of just the water content and compactive effort.

It is thus seen that the tabular form of the regression results provides a simple means by which the quality assurance engineer can take water content and density measurement from either a portion of the fill lift or statistically from the entire fill lift and determine expected unconfined shear strength and the expected variability in this magnitude. Furthermore, a method to evaluate the applicability of this table to other projects and soil types is conveniently given.

Table 12 represents only a portion of the values calculated for one water content variability magnitude (the variability found on the

Anderson test pad). The computer program presented in Appendix B will yield an entire listing for the same water content variability. To obtain listings for any other water content variabilities, the program lines preceeded by an "X" must be changed accordingly. A complete table for each roller type and soil condition would cover nearly 100 pages. Therefore, only a partial listing was presented here for illustrated purposes.

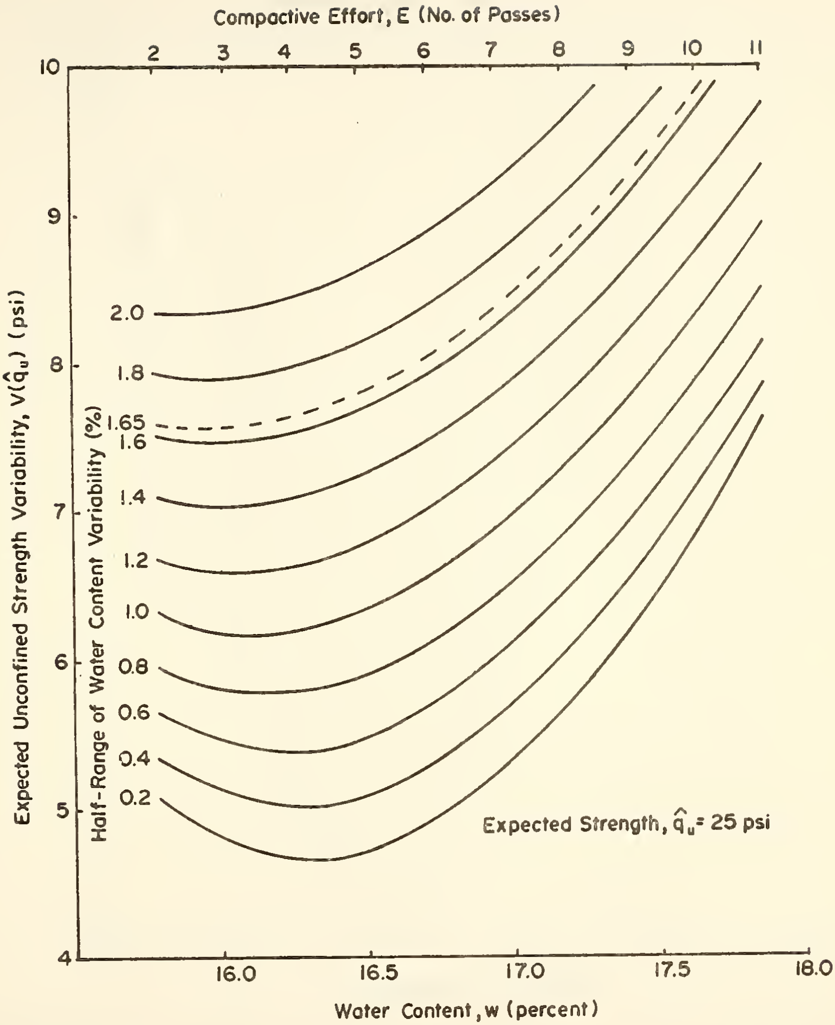


FIGURE 51 DESIGN GRAPH FOR 25 PSI EXPECTED STRENGTH: SHEEPSFOOT ROLLER, AS-COMPACTED

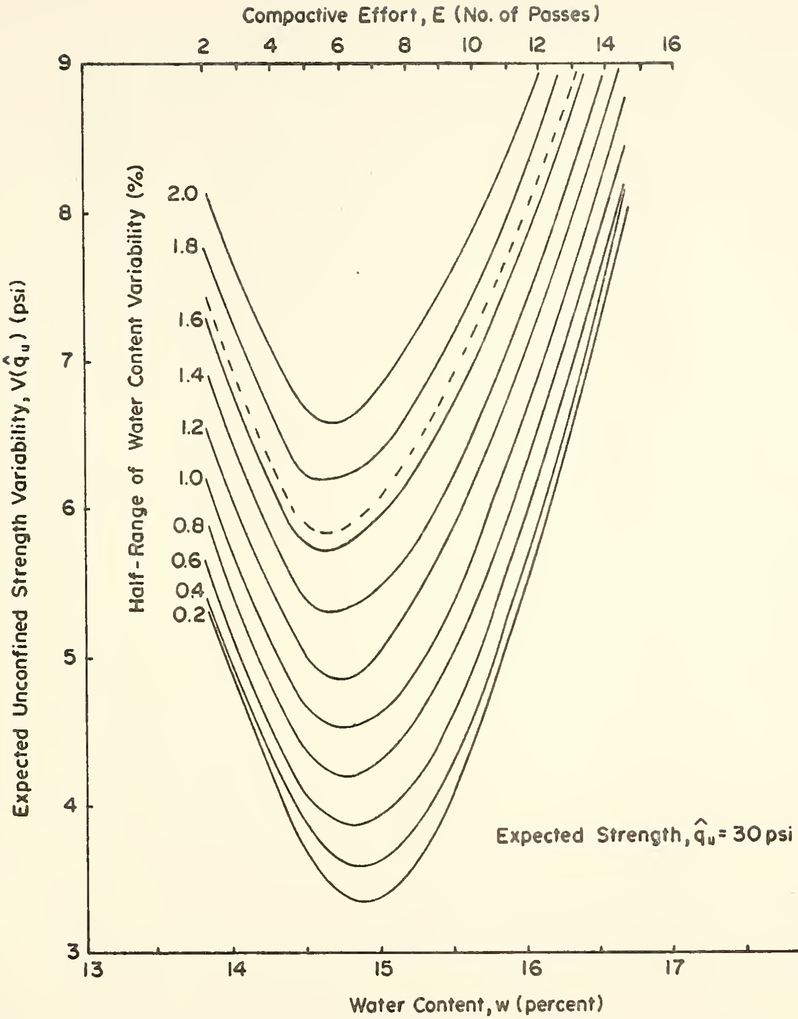


FIGURE 52 DESIGN GRAPH FOR 30 PSI EXPECTED STRENGTH: SHEEPSFOOT ROLLER, AS-COMPACTED

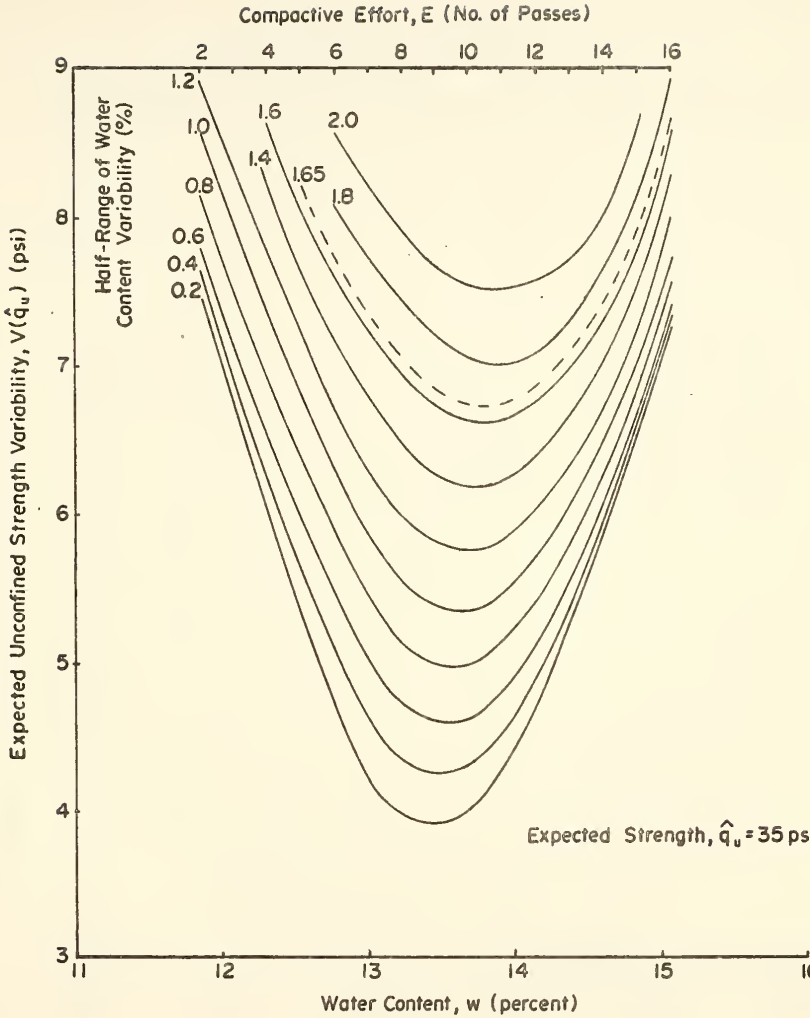


FIGURE 53 DESIGN GRAPH FOR 35 PSI EXPECTED STRENGTH: SHEEPSFOOT ROLLER, AS-COMPACTED

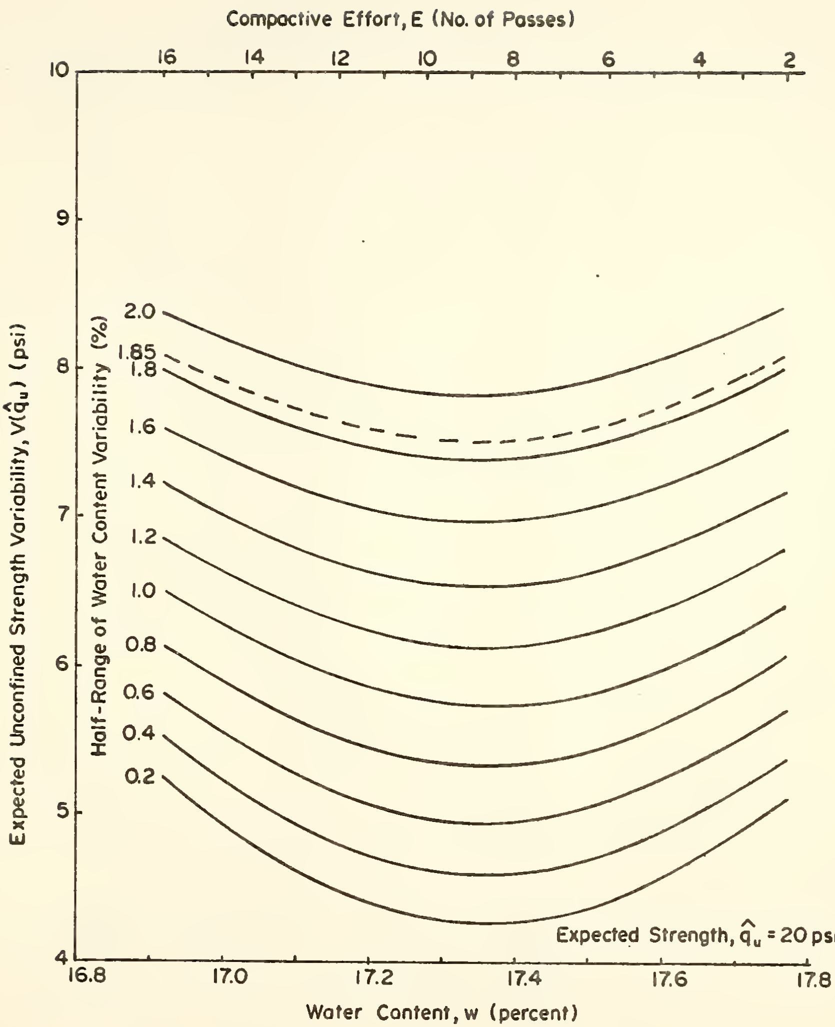


FIGURE 54 DESIGN GRAPH FOR 20 PSI EXPECTED STRENGTH: RUBBER-TIRED ROLLER, AS-COMPACTED

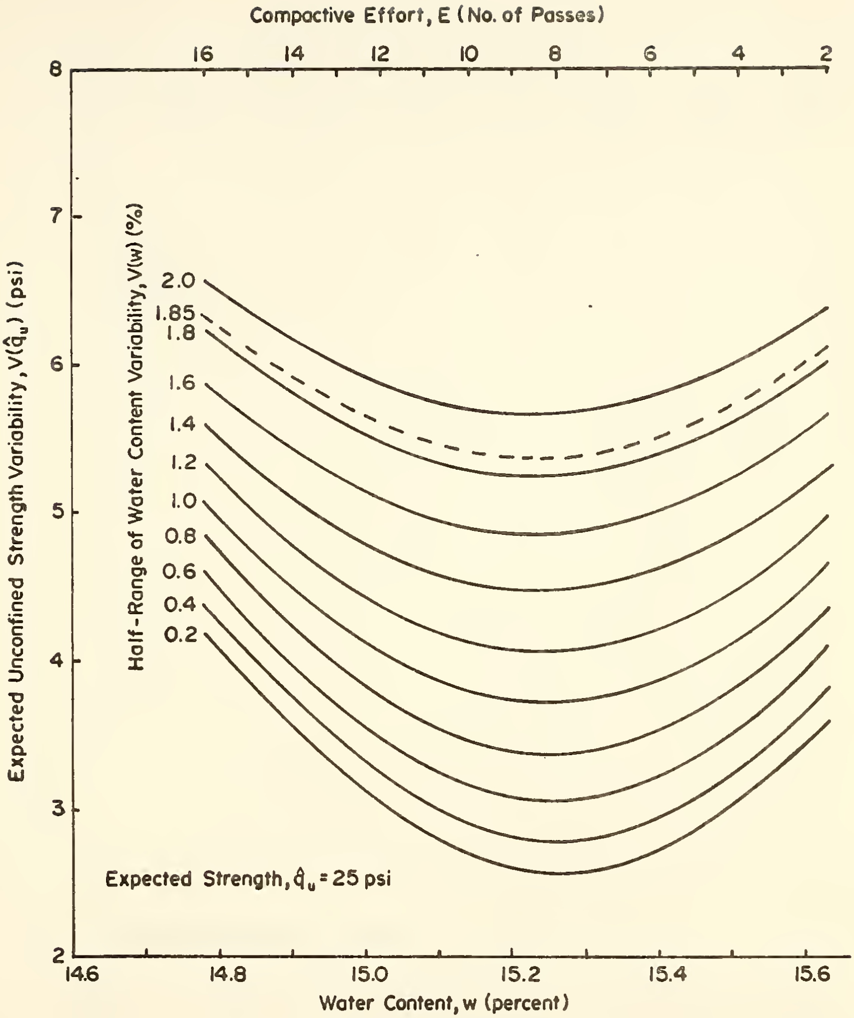


FIGURE 55 DESIGN GRAPH FOR 25 PSI EXPECTED STRENGTH: RUBBER-TIRED ROLLER, AS-COMPACTED

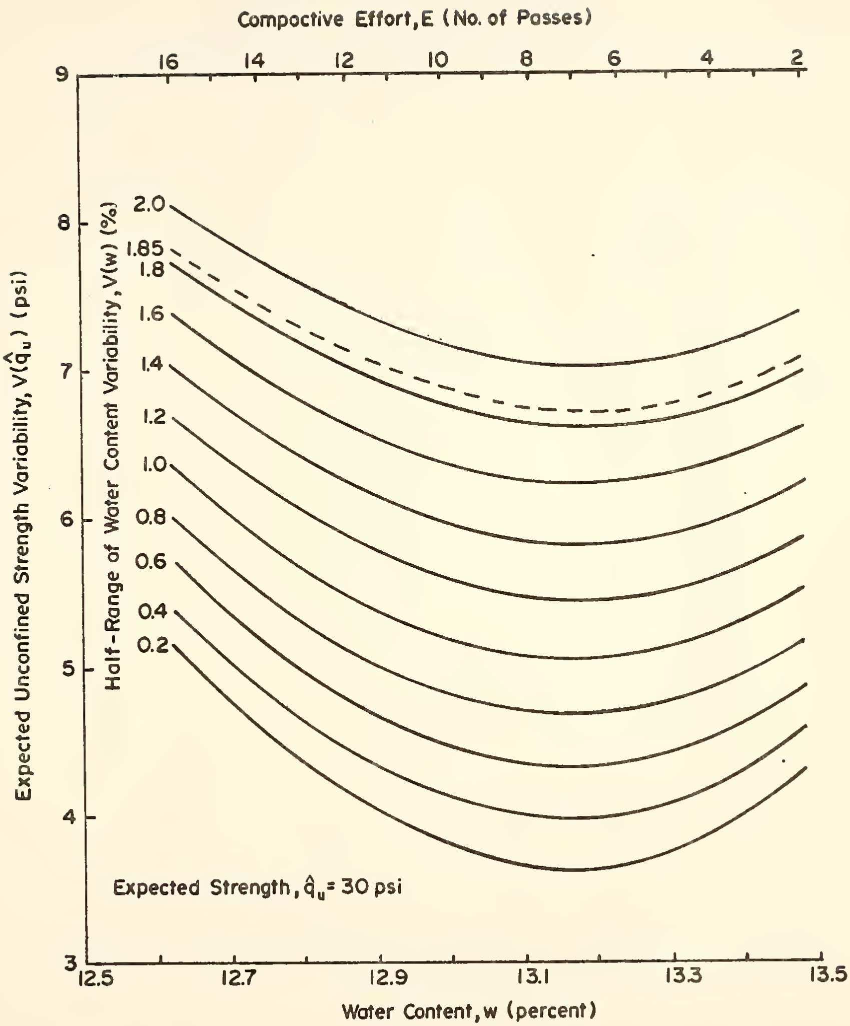


FIGURE 56 DESIGN GRAPH FOR 30 PSI EXPECTED STRENGTH: RUBBER-TIRED ROLLER, AS-COMPACTED

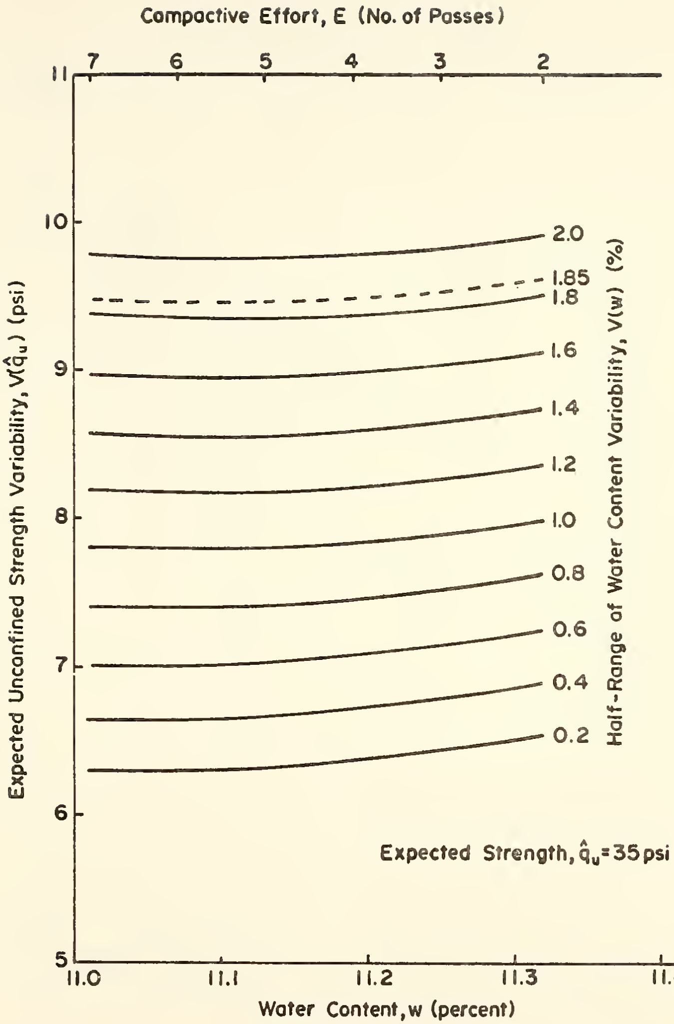


FIGURE 57 DESIGN GRAPH FOR 35 PSI EXPECTED STRENGTH: RUBBER-TIRED ROLLER, AS-COMPACTED

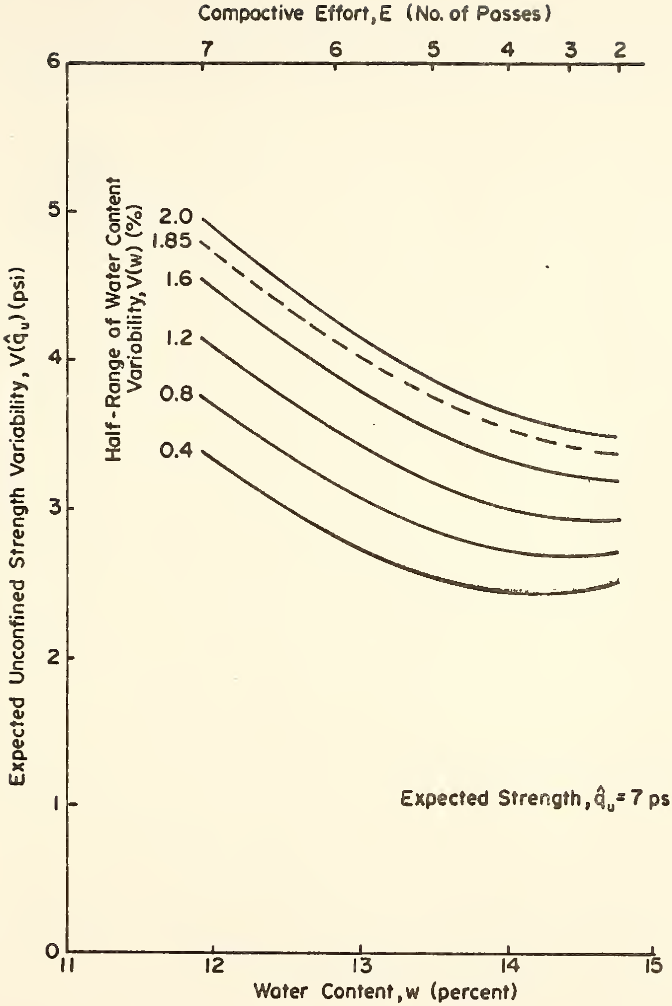


FIGURE 58 DESIGN GRAPH FOR 7 PSI EXPECTED STRENGTH: RUBBER-TIRED ROLLER, SOAKED

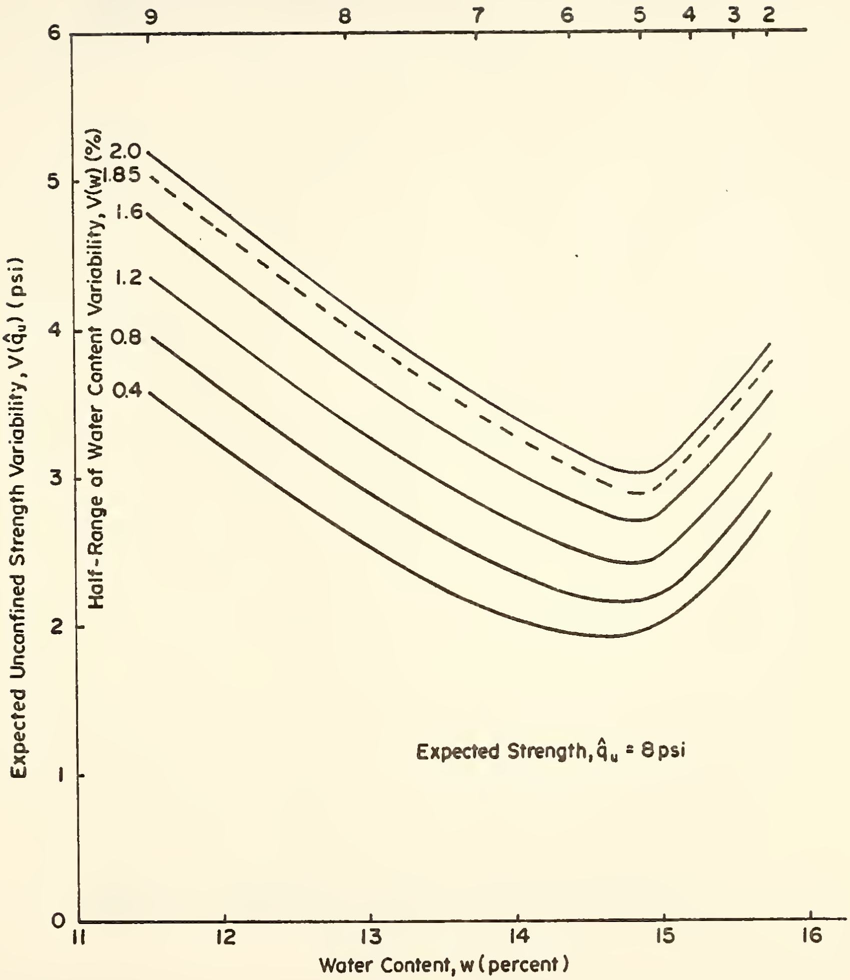


FIGURE 59 DESIGN GRAPH FOR 8 PSI EXPECTED STRENGTH: RUBBER-TIRED ROLLER, SOAKED

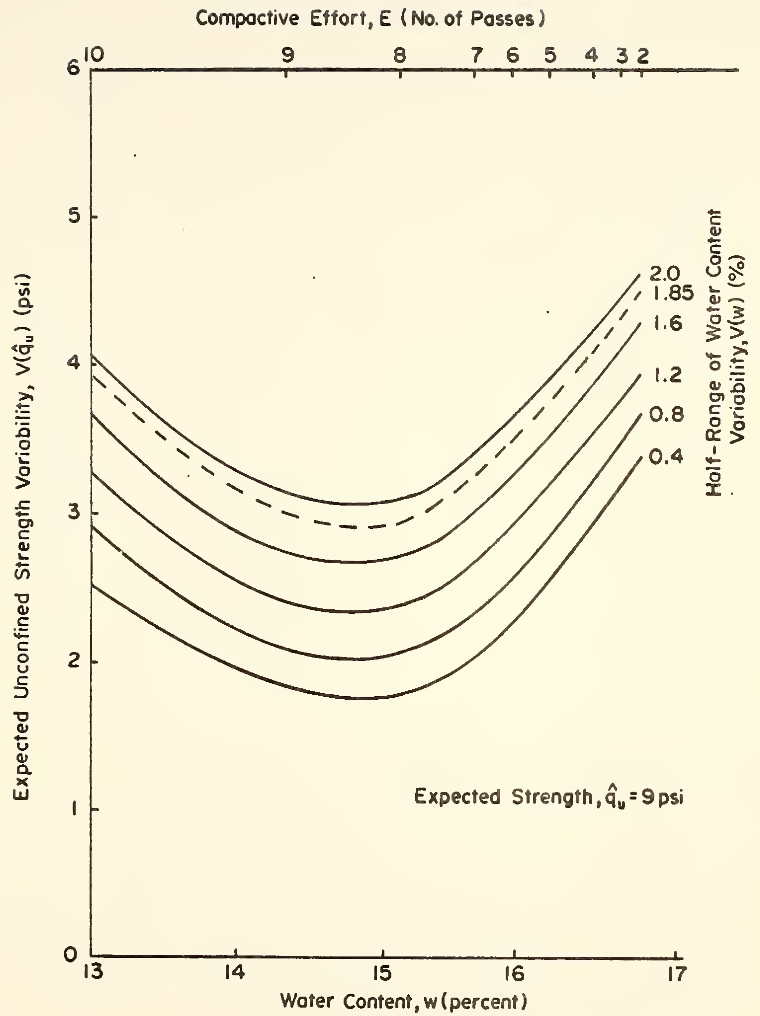


FIGURE 60 DESIGN GRAPH FOR 9 PSI EXPECTED STRENGTH: RUBBER-TIRED ROLLER, SOAKED

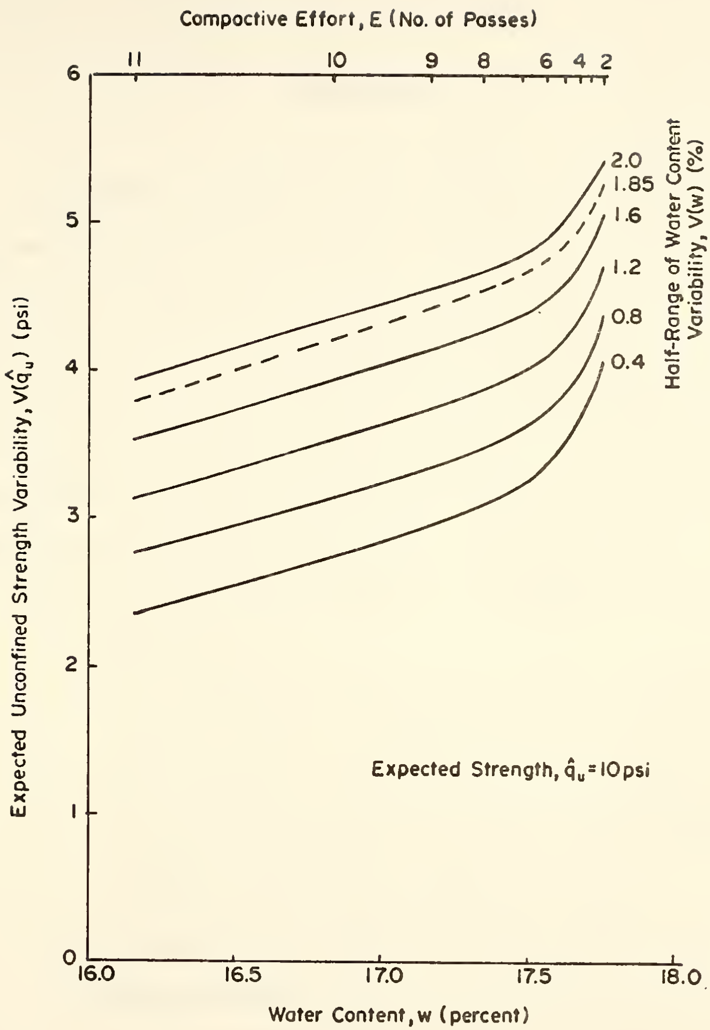


FIGURE 61 DESIGN GRAPH FOR 10 PSI EXPECTED STRENGTH: RUBBER-TIRED ROLLER, SOAKED

# CONCLUSIONS

This report culminated the work performed by Peterson (1975), Essigmann (1976), Scott (1977) and the author. The study included the statistical examination of a glacial silty-clay soil compacted in the field by both a rubber-tired roller or a sheepsfoot roller and the development of a correlation between a similar soil compacted in the laboratory and this field compacted soil. Given the constraints established by the project, specifically the wet of optimum water content limitation, the following conclusions could be made:

1) Soil compacted by a sheepsfoot roller exhibits the following relationships:
   a) The variable contributing most to the resultant as-compacted dry density magnitude is the water content.
   b) The variables contributing most to the resultant as-compacted unconfined strength magnitude are the water content and compactive effort.
2) Soil compacted by a rubber-tired roller exhibits the following relationships:
   a) The variables contributing most to the resultant as-compacted dry density magnitude are the compactive effort and the square of the water content.

b) The variables contributing most to the resultant soaked dry density magnitude are the compactive effort and the interaction between the compactive effort and the water content.

c) The variables contributing most to the resultant as-compacted unconfined shear strength magnitude are the water content, square of the water content and compactive effort. An increase in the water content or compactive effort causes a decrease in the shear strength.

d) The variables contributing most to the resultant soaked unconfined shear strength magnitude are the water content, compactive effort and interaction between the water content and compactive effort. The influence of either the water content or the compactive effort upon the shear strength is dependent upon the magnitude of the other independent variable.

3) The magnitude of the strength variability of both rollers is reduced if the water content variability is reduced.

4) The inherent variability in the compacted soil mass prevents consistently accurate measurement of the true construction quality by a one or two sample testing program. The average of a number of samples (possibly 5 to 7) must be used as a one point measurement of either the water content, dry density or unconfined shear strength.

5) Field strength response may be predicted by laboratory testing if a graphic superposition technique is used.

6) Large variabilities are expected and found in the dry density and unconfined strength parameters of a compacted soil. Future test pads should be strictly controlled to prevent large variabilities in the soil condition, testing procedure, compaction process and measured parameters.

## RECOMMENDATIONS FOR FUTURE STUDIES

1) Relationships should be developed that predict the field response of other soil parameters from laboratory testing. Consolidation, swell, confined shear strength, failure strains and moduli should be studied.

2) A study should be made to determine a quality assurance program for compacted soils that sufficiently reduce the risk of sampling in an unrepresentative manner.

3) Further work is necessary to determine how soil can be economically compacted to a more uniform consistency.

4) A study is necessary to determine the effective energy that is delivered to a soil during compaction by various currently used roller types , and by various modes of laboratory compaction.

5) A study should be made that identifies, separates and measures the various sources of variability found in compacted soils. Application of these results is necessary to improve current construction practices.

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APPENDICES

# APPENDIX A

Presentation of field and laboratory compacted sample data.

FIELD COMPACTED SAMPLE DATA - ANDERSON TEST PAD
SILTY CLAY A-4(7)

SHEEPSFOOT ROLLER - AS COMPACTED

| WATER CONTENT (PERCENT) | DRY DENSITY (PSI) | ENERGY (PASSES) | SHEAR STRENGTH (PSI) |
|---|---|---|---|
| 11.17 | 112.54 | 2. | 24.2 |
| 13.06 | 115.08 | 2. | 34.9 |
| 13.17 | 115.41 | 2. | 26.7 |
| 13.53 | 114.77 | 2. | 38.8 |
| 14.17 | 112.07 | 2. | 17.8 |
| 14.35 | 115.17 | 2. | 33.4 |
| 14.36 | 109.04 | 2. | 18.6 |
| 14.46 | 113.86 | 2. | 47.9 |
| 14.65 | 114.58 | 2. | 36.3 |
| 14.69 | 111.93 | 2. | 19.2 |
| 15.01 | 111.28 | 2. | 28.7 |
| 15.02 | 114.94 | 2. | 25.0 |
| 15.16 | 113.88 | 2. | 31.4 |
| 15.18 | 110.96 | 2. | 10.0 |
| 15.89 | 108.87 | 2. | 15.0 |
| 16.71 | 106.68 | 2. | 17.4 |
| 17.82 | 109.17 | 2. | 20.3 |
| 18.91 | 108.86 | 2. | 5.9 |
| 19.08 | 107.56 | 2. | 6.5 |
| 20.77 | 106.20 | 2. | 35.8 |
| 11.37 | 119.96 | 4. | 29.5 |
| 13.13 | 119.68 | 4. | 33.3 |
| 13.16 | 112.80 | 4. | 33.9 |
| 13.98 | 107.86 | 4. | 38.4 |
| 14.28 | 113.25 | 4. | 23.4 |
| 14.58 | 108.00 | 4. | 17.4 |
| 14.62 | 113.43 | 4. | 24.3 |
| 16.71 | 108.31 | 4. | 26.1 |
| 17.08 | 108.76 | 4. | 35.0 |
| 11.39 | 120.91 | 8. | 53.6 |
| 11.58 | 118.30 | 8. | 44.5 |
| 11.68 | 112.73 | 8. | 48.6 |
| 11.92 | 117.32 | 8. | 49.0 |
| 12.67 | 118.11 | 8. | 37.2 |
| 12.95 | 114.60 | 8. | 45.7 |
| 13.25 | 117.92 | 8. | 33.1 |
| 13.56 | 112.24 | 8. | 34.5 |
| 13.64 | 106.85 | 8. | 40.6 |
| 14.15 | 110.20 | 8. | 33.3 |
| 14.43 | 114.56 | 8. | 40.4 |
| 14.89 | 110.87 | 8. | 26.0 |
| 15.34 | 110.13 | 8. | 16.0 |
| 15.52 | 112.92 | 8. | 25.7 |

| WATER CONTENT (PERCENT) | DRY DENSITY (PSI) | ENERGY (PASSES) | SHEAR STRENGTH (PSI) |
|---|---|---|---|
| 15.82 | 111.26 | 8. | 23.0 |
| 16.23 | 108.95 | 8. | 33.3 |
| 16.52 | 111.94 | 8. | 13.9 |
| 17.95 | 109.36 | 8. | 18.2 |
| 18.11 | 106.59 | 8. | 26.3 |
| 10.96 | 118.27 | 16. | 31.9 |
| 11.01 | 122.72 | 16. | 52.0 |
| 12.73 | 119.15 | 16. | 47.0 |
| 12.80 | 117.39 | 16. | 44.0 |
| 12.91 | 107.04 | 16. | 19.0 |
| 13.01 | 118.29 | 16. | 38.1 |
| 13.12 | 105.12 | 16. | 44.6 |
| 13.35 | 114.83 | 16. | 48.2 |
| 13.37 | 113.37 | 16. | 38.3 |
| 14.03 | 114.50 | 16. | 24.9 |
| 14.15 | 118.38 | 16. | 39.7 |
| 14.93 | 109.99 | 16. | 32.8 |
| 15.87 | 112.56 | 16. | 41.5 |
| 16.28 | 110.48 | 16. | 23.3 |

RUBBER TIRE ROLLER - AS COMPACTED

| | | | |
|---|---|---|---|
| 12.86 | 114.17 | 2. | 36.0 |
| 13.01 | 116.45 | 2. | 27.0 |
| 13.06 | 116.29 | 2. | 29.1 |
| 14.37 | 113.40 | 2. | 23.8 |
| 14.77 | 113.49 | 2. | 22.9 |
| 14.88 | 109.71 | 2. | 30.1 |
| 14.88 | 110.29 | 2. | 23.6 |
| 14.96 | 115.50 | 2. | 32.5 |
| 15.26 | 112.51 | 2. | 24.2 |
| 15.29 | 111.17 | 2. | 23.2 |
| 15.51 | 109.69 | 2. | 25.0 |
| 15.58 | 107.08 | 2. | 26.3 |
| 16.10 | 109.36 | 2. | 17.2 |
| 16.16 | 110.69 | 2. | 22.7 |
| 16.64 | 110.07 | 2. | 10.2 |
| 17.81 | 109.64 | 2. | 25.7 |
| 17.99 | 109.44 | 2. | 20.7 |
| 11.09 | 112.34 | 4. | 20.9 |
| 12.47 | 118.06 | 4. | 36.6 |
| 12.53 | 114.20 | 4. | 32.8 |
| 12.62 | 114.06 | 4. | 34.4 |
| 12.85 | 114.53 | 4. | 22.2 |
| 13.90 | 115.96 | 4. | 33.2 |
| 14.11 | 116.26 | 4. | 21.9 |
| 14.31 | 110.01 | 4. | 43.3 |
| 14.37 | 108.07 | 4. | 23.5 |
| 14.83 | 114.31 | 4. | 25.2 |

| WATER CONTENT (PERCENT) | DRY DENSITY (PSI) | ENERGY (PASSES) | SHEAR STRENGTH (PSI) |
|---|---|---|---|
| 14.87 | 115.04 | 4. | 19.1 |
| 15.70 | 110.28 | 4. | 26.7 |
| 15.81 | 108.25 | 4. | 30.8 |
| 15.90 | 112.23 | 4. | 21.0 |
| 15.93 | 110.41 | 4. | 27.5 |
| 16.17 | 110.21 | 4. | 35.7 |
| 16.72 | 108.68 | 4. | 14.2 |
| 17.41 | 109.76 | 4. | 15.9 |
| 12.88 | 115.32 | 8. | 25.8 |
| 13.16 | 115.27 | 8. | 26.6 |
| 13.33 | 110.55 | 8. | 30.2 |
| 13.83 | 116.07 | 8. | 34.1 |
| 14.17 | 110.59 | 8. | 22.1 |
| 14.44 | 115.10 | 8. | 32.1 |
| 14.69 | 115.10 | 8. | 29.4 |
| 14.71 | 117.10 | 8. | 36.5 |
| 14.73 | 113.92 | 8. | 28.8 |
| 15.09 | 114.63 | 8. | 26.0 |
| 15.34 | 111.99 | 8. | 35.8 |
| 16.02 | 109.77 | 8. | 29.6 |
| 16.21 | 112.67 | 8. | 29.4 |
| 17.16 | 110.34 | 8. | 21.7 |
| 17.29 | 110.59 | 8. | 17.0 |
| 18.16 | 107.17 | 8. | 11.7 |
| 18.49 | 109.07 | 8. | 12.2 |
| 19.42 | 104.83 | 8. | 36.4 |
| 11.67 | 113.08 | 16. | 23.2 |
| 12.37 | 116.08 | 16. | 41.7 |
| 12.52 | 118.02 | 16. | 45.4 |
| 12.89 | 120.66 | 16. | 31.8 |
| 13.07 | 114.46 | 16. | 31.8 |
| 13.08 | 117.32 | 16. | 38.2 |
| 13.20 | 117.08 | 16. | 26.4 |
| 13.71 | 116.77 | 16. | 31.0 |
| 13.74 | 117.60 | 16. | 23.9 |
| 14.56 | 117.50 | 16. | 17.3 |
| 14.61 | 115.56 | 16. | 19.9 |
| 14.88 | 114.15 | 16. | 29.2 |
| 14.95 | 115.33 | 16. | 14.5 |
| 15.16 | 116.06 | 16. | 17.6 |
| 15.28 | 114.67 | 16. | 19.8 |
| 17.10 | 111.16 | 16. | 20.0 |
| 17.55 | 109.08 | 16. | 16.2 |
| 17.58 | 109.49 | 16. | 13.6 |
| 17.67 | 108.21 | 16. | 13.4 |

SHEEPSFOOT ROLLER - SOAKED

| WATER CONTENT (PERCENT) | DRY DENSITY (PSI) | ENERGY (PASSES) | SHEAR STRENGTH (PSI) |
|---|---|---|---|
| 13.30 | 110.44 | 2. | 4.2 |
| 11.44 | 110.72 | 4. | 7.3 |
| 12.87 | 109.28 | 4. | 6.4 |
| 15.16 | 108.43 | 4. | 5.7 |
| 12.43 | 113.67 | 8. | 6.6 |
| 14.04 | 112.70 | 8. | 7.8 |
| 14.28 | 113.20 | 8. | 8.1 |
| 14.75 | 110.50 | 8. | 7.9 |
| 15.88 | 111.54 | 8. | 13.1 |
| 16.38 | 108.68 | 8. | 6.8 |
| 13.12 | 115.41 | 16. | 8.4 |
| 13.45 | 111.07 | 16. | 10.3 |

RUBBER TIRE ROLLER - SOAKED

| | | | |
|---|---|---|---|
| 14.21 | 109.55 | 2. | 4.2 |
| 15.82 | 113.24 | 2. | 10.3 |
| 15.84 | 112.76 | 2. | 10.4 |
| 17.02 | 109.03 | 2. | 14.1 |
| 12.68 | 113.59 | 4. | 8.1 |
| 12.93 | 109.08 | 4. | 3.0 |
| 13.22 | 110.01 | 4. | 4.0 |
| 13.23 | 115.99 | 4. | 10.8 |
| 15.41 | 110.88 | 4. | 8.0 |
| 16.70 | 112.92 | 4. | 8.4 |
| 17.33 | 106.30 | 4. | 7.4 |
| 13.30 | 110.29 | 8. | 4.2 |
| 13.47 | 110.44 | 8. | 4.8 |
| 14.52 | 112.29 | 8. | 7.4 |
| 14.71 | 114.20 | 8. | 9.2 |
| 10.27 | 119.27 | 16. | 16.1 |
| 12.91 | 115.96 | 16. | 11.9 |
| 14.31 | 115.21 | 16. | 12.9 |
| 14.40 | 114.07 | 16. | 12.3 |
| 16.12 | 106.30 | 16. | 6.3 |
| 16.61 | 109.86 | 16. | 14.2 |
| 17.64 | 109.96 | 16. | 9.9 |

LABORATORY COMPACTED SAMPLE DATA - TESTED AS-COMPACTED
CLAYEY SILT A-4(5) FROM ESSIGMANN (1976)
15 BLOW PROCTOR

| WATER CONTENT (PERCENT) | DRY DENSITY (PCF) | ENERGY (RATIO) | SHEAR STRENGTH (PSI) |
|---|---|---|---|
| 7.04 | 106.50 | 1.00 | 13.5 |
| 7.09 | 107.60 | 1.00 | 15.1 |
| 7.10 | 107.20 | 1.00 | 12.3 |
| 8.20 | 110.30 | 1.00 | 16.0 |
| 8.20 | 108.40 | 1.00 | 14.1 |
| 8.30 | 111.30 | 1.00 | 19.2 |
| 9.50 | 113.10 | 1.00 | 18.0 |
| 9.50 | 115.70 | 1.00 | 27.0 |
| 9.50 | 110.60 | 1.00 | 16.2 |
| 10.20 | 117.10 | 1.00 | 21.2 |
| 10.30 | 116.20 | 1.00 | 26.3 |
| 10.30 | 116.30 | 1.00 | 22.9 |
| 11.10 | 115.20 | 1.00 | 16.2 |
| 11.30 | 119.80 | 1.00 | 35.6 |
| 11.40 | 119.70 | 1.00 | 33.4 |
| 11.40 | 119.60 | 1.00 | 34.8 |
| 12.00 | 120.80 | 1.00 | 31.4 |
| 12.10 | 119.40 | 1.00 | 30.5 |
| 12.20 | 120.70 | 1.00 | 27.5 |
| 12.80 | 121.60 | 1.00 | 27.9 |
| 12.80 | 120.90 | 1.00 | 26.5 |
| 12.90 | 120.90 | 1.00 | 27.5 |
| 13.70 | 121.00 | 1.00 | 16.0 |
| 13.90 | 119.90 | 1.00 | 11.9 |
| 13.90 | 119.20 | 1.00 | 15.2 |
| 15.20 | 117.90 | 1.00 | 6.8 |
| 15.30 | 117.60 | 1.00 | 7.1 |
| STANDARD PROCTOR | | | |
| 7.60 | 110.10 | 1.67 | 23.5 |
| 7.90 | 111.60 | 1.67 | 19.5 |
| 7.90 | 110.10 | 1.67 | 20.8 |
| 9.00 | 113.60 | 1.67 | 32.3 |
| 9.10 | 119.50 | 1.67 | 51.7 |
| 9.20 | 119.10 | 1.67 | 49.0 |
| 9.30 | 117.10 | 1.67 | 31.4 |
| 9.50 | 118.70 | 1.67 | 40.5 |
| 9.60 | 117.80 | 1.67 | 42.2 |
| 10.30 | 121.60 | 1.67 | 45.0 |
| 10.30 | 123.60 | 1.67 | 43.0 |
| 10.50 | 124.30 | 1.67 | 54.6 |
| 10.90 | 124.40 | 1.67 | 49.5 |
| 10.90 | 122.70 | 1.67 | 54.5 |
| 10.90 | 122.30 | 1.67 | 52.2 |

| WATER CONTENT (PERCENT) | DRY DENSITY (PCF) | ENERGY (RATIO) | SHEAR STRENGTH (PSI) |
|---|---|---|---|
| 11.60 | 122.20 | 1.67 | 43.3 |
| 11.60 | 123.90 | 1.67 | 45.1 |
| 11.70 | 122.70 | 1.67 | 40.5 |
| 11.70 | 124.30 | 1.67 | 48.6 |
| 11.90 | 122.70 | 1.67 | 38.0 |
| 12.30 | 123.80 | 1.67 | 36.0 |
| 12.80 | 122.40 | 1.67 | 29.0 |
| 13.50 | 119.20 | 1.67 | 18.2 |
| 13.50 | 119.10 | 1.67 | 17.0 |
| 13.60 | 119.00 | 1.67 | 18.5 |
| 13.70 | 120.10 | 1.67 | 16.7 |
| 13.70 | 120.50 | 1.67 | 14.8 |
| 13.90 | 119.30 | 1.67 | 14.5 |
| 14.90 | 115.90 | 1.67 | 8.3 |
| 15.00 | 117.30 | 1.67 | 7.3 |
| 15.50 | 116.90 | 1.67 | 5.2 |
| 15.50 | 116.20 | 1.67 | 5.8 |
| 15.50 | 116.30 | 1.67 | 5.0 |
| 15.70 | 114.80 | 1.67 | 4.8 |
| 15.70 | 114.50 | 1.67 | 6.1 |
| 15.90 | 115.80 | 1.67 | 3.8 |

25 BLOW PROCTOR

| | | | |
|---|---|---|---|
| 7.12 | 121.50 | 4.56 | 47.5 |
| 7.21 | 119.90 | 4.56 | 54.9 |
| 8.09 | 123.40 | 4.56 | 69.6 |
| 8.10 | 121.60 | 4.56 | 61.0 |
| 8.21 | 124.80 | 4.56 | 88.1 |
| 8.24 | 121.70 | 4.56 | 61.1 |
| 8.37 | 122.50 | 4.56 | 69.5 |
| 9.84 | 126.30 | 4.56 | 77.3 |
| 9.80 | 126.00 | 4.56 | 56.1 |
| 9.94 | 125.60 | 4.56 | 68.4 |
| 9.94 | 124.70 | 4.56 | 67.8 |
| 9.96 | 124.80 | 4.56 | 72.0 |
| 9.99 | 125.50 | 4.56 | 80.0 |
| 10.03 | 125.00 | 4.56 | 69.2 |
| 10.12 | 125.20 | 4.56 | 69.0 |
| 10.16 | 126.90 | 4.56 | 75.1 |
| 10.81 | 124.60 | 4.56 | 58.6 |
| 10.81 | 123.90 | 4.56 | 50.0 |
| 10.84 | 125.90 | 4.56 | 67.3 |
| 13.23 | 121.40 | 4.56 | 17.6 |
| 13.44 | 120.40 | 4.56 | 16.5 |
| 13.50 | 119.10 | 4.56 | 15.8 |

MODIFIED PROCTOR

| | | | |
|---|---|---|---|
| 9.40 | 128.90 | 7.61 | 100.0 |
| 9.60 | 127.80 | 7.61 | 76.3 |
| 10.10 | 126.70 | 7.61 | 90.7 |
| 10.20 | 127.60 | 7.61 | 75.1 |
| 10.30 | 130.30 | 7.61 | 70.7 |
| 10.50 | 124.50 | 7.61 | 54.3 |
| 10.50 | 128.00 | 7.61 | 64.0 |
| 10.50 | 126.50 | 7.61 | 68.0 |
| 11.00 | 127.70 | 7.61 | 70.8 |
| 11.10 | 126.80 | 7.61 | 58.8 |
| 11.20 | 125.70 | 7.61 | 52.0 |
| 12.00 | 122.00 | 7.61 | 34.5 |
| 12.00 | 123.00 | 7.61 | 34.0 |
| 12.00 | 125.10 | 7.61 | 49.1 |
| 12.00 | 121.60 | 7.61 | 45.8 |
| 12.10 | 122.60 | 7.61 | 47.2 |
| 12.20 | 122.10 | 7.61 | 34.5 |
| 12.50 | 120.50 | 7.61 | 28.2 |
| 12.50 | 123.30 | 7.61 | 29.3 |
| 12.80 | 122.80 | 7.61 | 24.7 |

LABORATORY COMPACTED SAMPLE DATA - TESTED SOAKED
CLAYEY SILT A-4(5) FROM SCOTT (1977)
15 BLOW PROCTOR

| | | | |
|---|---|---|---|
| 8.50 | 110.40 | 1.00 | .5 |
| 8.70 | 110.00 | 1.00 | 1.0 |
| 9.00 | 113.80 | 1.00 | .3 |
| 9.40 | 111.60 | 1.00 | .8 |
| 9.40 | 114.20 | 1.00 | 1.5 |
| 9.70 | 114.70 | 1.00 | .9 |
| 9.90 | 113.00 | 1.00 | .5 |
| 9.90 | 114.60 | 1.00 | .3 |
| 9.90 | 120.60 | 1.00 | 3.0 |
| 10.00 | 118.10 | 1.00 | .9 |
| 10.30 | 119.70 | 1.00 | 2.5 |
| 10.30 | 122.20 | 1.00 | 4.0 |
| 10.80 | 120.80 | 1.00 | 2.6 |
| 10.80 | 121.90 | 1.00 | 4.6 |
| 11.10 | 122.00 | 1.00 | 6.8 |
| 11.20 | 124.60 | 1.00 | 7.1 |
| 11.30 | 122.50 | 1.00 | 5.0 |
| 11.50 | 122.40 | 1.00 | 12.3 |
| 11.50 | 123.00 | 1.00 | 8.5 |
| 11.60 | 122.40 | 1.00 | 8.9 |
| 11.60 | 122.90 | 1.00 | 12.8 |
| 12.10 | 121.00 | 1.00 | 4.5 |
| 12.10 | 121.70 | 1.00 | 14.0 |
| 12.50 | 122.20 | 1.00 | 8.0 |
| 12.50 | 123.10 | 1.00 | 5.5 |

| WATER CONTENT (PERCENT) | DRY DENSITY (PCF) | ENERGY (RATIO) | SHEAR STRENGTH (PSI) |
|---|---|---|---|
| 12.60 | 124.00 | 1.00 | 5.1 |
| 12.70 | 119.50 | 1.00 | 4.2 |
| 12.70 | 123.50 | 1.00 | 9.7 |
| 12.70 | 125.70 | 1.00 | 8.0 |
| 12.80 | 124.80 | 1.00 | 6.7 |
| 13.40 | 120.20 | 1.00 | 4.5 |
| 13.80 | 119.30 | 1.00 | 4.0 |
| 13.80 | 123.60 | 1.00 | 5.6 |
| 13.90 | 120.60 | 1.00 | 4.7 |
| 13.90 | 121.30 | 1.00 | 4.7 |

STANDARD PROCTOR

| | | | |
|---|---|---|---|
| 8.80 | 114.90 | 1.67 | .5 |
| 8.80 | 116.50 | 1.67 | .7 |
| 8.90 | 115.40 | 1.67 | .6 |
| 8.90 | 118.60 | 1.67 | 1.7 |
| 8.90 | 118.70 | 1.67 | 3.8 |
| 9.10 | 119.00 | 1.67 | 1.3 |
| 9.60 | 120.40 | 1.67 | 1.9 |
| 9.80 | 121.50 | 1.67 | 1.4 |
| 9.80 | 123.90 | 1.67 | 1.9 |
| 9.90 | 117.90 | 1.67 | 2.5 |
| 9.90 | 121.30 | 1.67 | 2.8 |
| 10.60 | 123.50 | 1.67 | 12.4 |
| 10.60 | 124.10 | 1.67 | 11.0 |
| 10.70 | 123.00 | 1.67 | 5.7 |
| 10.90 | 122.50 | 1.67 | 15.5 |
| 10.90 | 123.30 | 1.67 | 4.3 |
| 11.00 | 121.60 | 1.67 | 2.4 |
| 11.00 | 122.30 | 1.67 | 9.5 |
| 11.10 | 123.00 | 1.67 | 8.7 |
| 11.10 | 124.40 | 1.67 | 7.7 |
| 11.30 | 123.20 | 1.67 | 8.0 |
| 11.50 | 122.00 | 1.67 | 15.0 |
| 11.50 | 122.00 | 1.67 | 19.0 |
| 11.60 | 121.30 | 1.67 | 16.2 |
| 12.30 | 120.00 | 1.67 | 12.3 |
| 12.30 | 120.90 | 1.67 | 12.2 |
| 12.80 | 117.50 | 1.67 | 8.2 |
| 12.80 | 119.70 | 1.67 | 6.9 |
| 13.00 | 121.20 | 1.67 | 6.9 |
| 13.40 | 118.10 | 1.67 | 6.7 |
| 13.40 | 119.10 | 1.67 | 6.0 |
| 13.50 | 120.00 | 1.67 | 7.8 |
| 13.50 | 120.70 | 1.67 | 6.9 |
| 13.70 | 117.50 | 1.67 | 6.2 |
| 13.80 | 120.00 | 1.67 | 6.3 |
| 13.80 | 120.90 | 1.67 | 6.7 |

| WATER CONTENT (PERCENT) | DRY DENSITY (PCF) | ENERGY (RATIO) | SHEAR STRENGTH (PSI) |
|---|---|---|---|
| 14.00 | 116.10 | 1.67 | 4.3 |
| 14.00 | 120.90 | 1.67 | 2.2 |
| 14.10 | 118.00 | 1.67 | 8.2 |
| 14.10 | 119.80 | 1.67 | 5.6 |
| 25 BLOW PROCTOR | | | |
| 8.10 | 124.10 | 4.56 | 3.7 |
| 8.10 | 125.10 | 4.56 | 2.1 |
| 8.70 | 123.20 | 4.56 | 9.5 |
| 8.70 | 125.40 | 4.56 | 6.6 |
| 8.70 | 125.70 | 4.56 | 8.1 |
| 8.80 | 124.60 | 4.56 | 10.7 |
| 8.00 | 125.40 | 4.56 | 12.1 |
| 9.10 | 127.70 | 4.56 | 25.0 |
| 9.50 | 129.20 | 4.56 | 21.0 |
| 9.50 | 125.20 | 4.56 | 18.1 |
| 9.50 | 129.40 | 4.56 | 31.7 |
| 9.70 | 126.70 | 4.56 | 26.4 |
| 9.70 | 127.00 | 4.56 | 20.3 |
| 9.80 | 125.80 | 4.56 | 28.2 |
| 10.00 | 127.80 | 4.56 | 35.9 |
| 10.00 | 128.00 | 4.56 | 26.1 |
| 10.50 | 123.10 | 4.56 | 16.0 |
| 10.50 | 123.10 | 4.56 | 20.5 |
| 10.50 | 124.10 | 4.56 | 12.5 |
| 10.50 | 124.20 | 4.56 | 17.7 |
| 10.50 | 124.50 | 4.56 | 14.6 |
| 10.90 | 123.00 | 4.56 | 12.0 |
| 10.90 | 125.10 | 4.56 | 12.1 |
| 11.10 | 125.10 | 4.56 | 37.9 |
| 11.60 | 124.20 | 4.56 | 18.1 |
| 11.70 | 123.00 | 4.56 | 9.6 |
| 11.70 | 127.60 | 4.56 | 10.2 |
| 11.90 | 120.50 | 4.56 | 16.3 |
| 11.90 | 121.50 | 4.56 | 20.0 |
| 11.90 | 122.90 | 4.56 | 10.9 |
| 12.10 | 124.60 | 4.56 | 10.4 |
| 12.10 | 127.40 | 4.56 | 9.4 |
| 12.30 | 126.40 | 4.56 | 8.5 |
| 12.70 | 115.40 | 4.56 | 10.7 |
| 12.70 | 124.90 | 4.56 | 12.7 |
| 13.20 | 120.90 | 4.56 | 8.8 |
| 13.30 | 119.70 | 4.56 | 8.2 |
| 13.30 | 124.50 | 4.56 | 5.1 |
| 13.40 | 122.70 | 4.56 | 9.2 |
| 13.40 | 123.20 | 4.56 | 5.0 |
| 13.40 | 124.20 | 4.56 | 6.3 |

# APPENDIX B

An explanation of the strength variability formula.

An example of the formula's use.

Computer program used for strength variability calculations:

a) sheepsfoot roller: as-compacted,

b) rubber-tired roller: as-compacted,

c) rubber-tired roller: soaked.

An explanation of the strength variability formula

$V(\hat{q}_u)=\lambda s\sqrt{X_p'[X'X]^{-1}X_p}$ = Unconfined Shear Strength Variability defined for one set of water content, dry density and compactive effort values for each calculation.

$X_p$=column vector containing the values of the independent variables for which the evaluation of $V(\hat{q}_u)$ is sought.

$X_p'=X_p^t=X_p$ transposed.

s=square root of the strength regression model mean square error ($\sqrt{MSE}$). The value is obtained from the REGRESSION computer program ANOVA table.

λ=appropriate t-statistic. Its value is dependent upon the number of samples used in the REGRESSION program and upon the level of confidence the design engineer feels is necessary for the given project. As the number of samples increases, the value of λ decreases, and the greater the degree of confidence the engineer desires for a given number of samples, the higher the value of λ must be. The level of confidence is synonomous with the probability of not making a statistical Type I error. The probability of making a Type I error is called the alpha (α) level. Therefore, the confidence level is equal to 1-α. Within the context of this investigation, a Type I error is the rejection of a fill lift or embankment section due to inadequate control test results when in fact, the strength of the

lift of section is adequate. Obviously, both the engineer and the contractor must keep the $\alpha$ level low as undue compaction costs will ensue if many good embankment sections are deemed unsuitable.

$$[X'X]^{-1} = \begin{bmatrix} \frac{1}{n} + \frac{1}{s^2} R'VR & -\frac{1}{s^2} R'V \\ -\frac{1}{s^2} VR & \frac{1}{s^2} V \end{bmatrix}$$

Where:

n=number of samples tested in developing the regression or predictive model.

s=as defined above.

R=column vector of the means of the independent variables used in the regression model; obtained from the output of REGRESSION.

$R'=R^t$=R transposed.

V=symmetric variance/cocariance matrix of the regression model.

Note: The expected dry density variability is obtained in a similar manner. For the purposes of the following example, the dry density variability will be assumed as given. For a more complete understanding of the calculations involved in the dry density variability determinations, review the computor program listed in the following section.

An example of the formula's use

GIVEN: a) Statistical prediction model for the dry density of a silty clay soil compacted by a rubber-tired roller with samples tested in an as-compacted condition.

$\hat{\gamma}_d = 122.46 - 0.048\ w^2 + 0.169\ E$ in PCF

b) Statistical prediction model for the unconfined shear strength of the same soil as in "a" above.

$\hat{q}_u = -2.224\ w - 0.155\ E + 0.734\hat{\gamma}_d + 51.91$ in PSI

c) Computer output with one-way ANOVA table obtained from the strength REGRESSION run.

DESIRED: The design engineer having reviewed compaction curves, specifies that this soil is to be compacted at a 13% water content using 7 passes of this particular type of compaction equipment. To perform a stability analysis, he wishes to know the variability in the strength that can be expected from this process. With this knowledge, an appropriate factor of safety may be applied to the lowest expected unconfined strength.

ANALYSIS:

a) $\hat{q}_u - V(\hat{q}_u)$ = lowest expected strength under the given compaction conditions.

b) $V(\hat{q}_u) = \lambda s \sqrt{X_p'[X'X]^{-1}X_p}$ where four calculations of $V(\hat{q}_u)$ must be made for the following conditions:

| Trial | E | w | $\gamma_d$ | $V(\hat{q}_u)$ |
|---|---|---|---|---|
| 1 | 7 | 13 + 1.85 | $\hat{\gamma}_d + V(\hat{\gamma}_d)$ | $V(\hat{q}_u)_1$ |
| 2 | 7 | 13 - 1.85 | $\hat{\gamma}_d + V(\hat{\gamma}_d)$ | $V(\hat{q}_u)_2$ |
| 3 | 7 | 13 + 1.85 | $\hat{\gamma}_d - V(\hat{\gamma}_d)$ | $V(\hat{q}_u)_3$ |
| 4 | 7 | 13 - 1.85 | $\hat{\gamma}_d - V(\hat{\gamma}_d)$ | $V(\hat{q}_u)_4$ |

SOLUTION:

a) $\hat{\gamma}_d = 122.46 - 0.048(13) + 0.169(7) = 115.53$ PCF

b) $V(\hat{\gamma}_d) = 1.46$ PCF  See the computer program for a detailed explanation on how this number was derived.

c) $\hat{q}_u = 51.91 - 2.224(13) - 0.155(7) + 0.0734(115.53) =$ 30.4 PSI

d) From the computer output of the regression run: n = 60

n = 60

s = 6.458

$R' = (\bar{w}\ \bar{E}\ \bar{\gamma}_d) = 14.95 \quad 7.90 \quad 113.01)$ These averages were computed from the data used in establishing the prediction modes. Therefore, they do not change from soil to soil if the prediction model is used for each soil.

$$V = \begin{vmatrix} 0.58626 & -0.03133 & 0.24043 \\ -0.03133 & 0.02763 & -0.02696 \\ 0.24043 & -0.02696 & 0.16665 \end{vmatrix}$$

$$[X'X]^{-1} = \begin{vmatrix} 72.378 & -0.85571 & 0.07905 & -0.5326 \\ -0.85571 & 0.01406 & -0.00075 & 0.00577 \\ 0.07905 & -0.00075 & 0.00063 & -0.00065 \\ -0.53267 & 0.00577 & -0.00065 & 0.00400 \end{vmatrix}$$

e) $X'_p = (1.0 \quad 14.85 \quad 7.0 \quad 114.07)$ This corresponds to the value of w+1.85, E and $\hat{\gamma}_d + V(\hat{\gamma}_d)$ for which the strength variability is desired. The constant "1.0" is included within this vector because it is the identy factor for the constant term 51.91 in the strength prediction equation. The sequence of these numbers must correspond to the sequence within the regression equation.

f) The desired level of confidence chosen for this investigation is 95%. Since there exists 4 endpoints to the problem, $(V(\hat{q}_u))$ must be calculated for the 4 combinations of w and $\hat{\gamma}_d$ shown on page 56), and because the overall alpha level is not to exceed 0.05, the tolerable alpha level for each of the 4 endpoint calculations is 0.05/4 = 0.0125. This is called multiplicity of endpoints and is synonomous with a two-tailed t-statistic by which the desired alpha level for each tail would be 0.05/2. To develop the regression equation, 4 degrees of freedom are lost.

Therefore, the appropriate t-statistic is:

$$t_{(\alpha/4,n-4)} = t_{(0.05/4,60-4)} = t_{(0.0125,56)} =$$

$$2.581 = \lambda$$

Upon substitution into the $V(\hat{q}_u)$ equation, the strength variability is equal to 2.45 PSI. The other three end-points must be calculated in a similar manner. The results are shown below.

| E | w | $\hat{\gamma}_d$ | $V(\hat{q}_u)$ |
|---|---|---|---|
| 7 | 14.85 | 116.99 | 4.74 |
| 7 | 11.15 | 116.99 | 5.49 |
| 7 | 14.85 | 114.07 | 2.45 |
| 7 | 11.15 | 114.07 | 6.95 |

As the design engineer is concerned with the stability of the embankment when the strength is lowest, the largest value of $V(\hat{q}_u)$ is used to represent the expected variability in the unconfined strength. Therefore, for the silty clay soil compacted at a 13% water content and a 7 pass compaction effort by the defined rubber-tired roller, the expected variability of the unconfined shear strength is 6.95 PSI.

COMPUTER PROGRAM USED FOR STRENGTH VARIABILITY CALCULATIONS:SFR-AC

```
C  XD=VECTOR OF INDEPENDENT VARIABLES FOR DRY DENSITY
C  XXD=[X'X]^-1  FOR DRY DENSITY
C  LAMDAD=λ FOR DRY DENSITY
C  SD=SQUARE ROOT OF THE MEAN SQUARE ERROR FOR DRY DENSITY
C  XP=VECTOR OF INDEPENDENT VARIABLES FOR STRENGTH
C  XX=[X'X]^-1  FOR UNCONFINED COMPRESSIVE STRENGTH
C  BETA=CONSTANT AND VARIABLE COEFFICIENTS OF THE STRENGTH EQUATION
C  LAMDA=λ FOR UNCONFINED COMPRESSIVE STRENGTH
C  S=SQUARE ROOT OF THE MEAN SQUARE ERROR FOR STRENGTH
C
      REAL LAMDA,MA,MB,MC,MD,LS,LAMDAD,LDS
      DIMENSION XP(4),XX(4,4),BETA(4),MA(4),MB(4),MC(4),MD(4),XXD(2,2),X
     1DPXXA(2),XDPXXB(2),XD(2)
C
C  READ NECESSARY INFORMATION
C
      READ(1,900)XXD
      READ(1,1015)LAMDAD,SD
      READ (1,1005)BETA
      READ (1,1010)XX
      READ(1,1015)LAMDA,S
      LS=LAMDA*S
      LDS=LAMDAD*SD
      PRINT 100
      XD(1)=1.0
      XP(1)=1.0
      GRAPH=2.
      DO 120 K=1,15
      XP(3)=GRAPH
      XAH=11.
      DO 105 J=1,71
      XD(2)=XAH
      XP(2)=XAH
C
C  2 ENDPOINT DENSITY VARIABILITY CALCULATIONS
C
      XD(2)=XD(2)-1.65
      XDPXXA(1)=XD(1)*XXD(1,1)+XD(2)*XXD(2,1)
      XDPXXA(2)=XD(1)*XXD(1,2)+XD(2)*XXD(2,2)
      VDD1=XDPXXA(1)*XD(1)+XDPXXA(2)*XD(2)
      VDD1=SQRT(VDD1)
      D1=SD*VDD1
      XD(2)=XD(2)+3.3
      XDPXXB(1)=XD(1)*XXD(1,1)+XD(2)*XXD(2,1)
      XDPXXB(2)=XD(1)*XXD(1,2)+XD(2)*XXD(2,2)
      VDD2=XDPXXB(1)*XD(1)+XDPXXB(2)*XD(2)
      VDD2=SQRT(VDD2)
      D2=SD*VDD2
      XD(2)=XD(2)-1.65
      IF(D2.GE.D1)GO TO 301
      D=D1
      GO TO 302
  301 D=D2
```

```
C
C  REGRESSION EQUATION EVALUATIONS FOR DENSITY AND STRENGTH
C
  302 XP(4)=133.43-(XP(2)*1.398)
      Q=XP(1)*BETA(1)+XP(2)*BETA(2)+XP(3)*BETA(3)+XP(4)*BETA(4)
C
C  4 ENDPOINT STRENGTH VARIABILITY CALCULATION
C
      XP(2)=XP(2)-1.65
      XP(4)=XP(4)-D
      MA(1)=XP(1)*XX(1,1)+XP(2)*XX(2,1)+XP(3)*XX(3,1)+XP(4)*XX(4,1)
      MA(2)=XP(1)*XX(1,2)+XP(2)*XX(2,2)+XP(3)*XX(3,2)+XP(4)*XX(4,2)
      MA(3)=XP(1)*XX(1,3)+XP(2)*XX(2,3)+XP(3)*XX(3,3)+XP(4)*XX(4,3)
      MA(4)=XP(1)*XX(1,4)+XP(2)*XX(2,4)+XP(3)*XX(3,4)+XP(4)*XX(4,4)
      V1=MA(1)*XP(1)+MA(2)*XP(2)+MA(3)*XP(3)+MA(4)*XP(4)
      V1=SQRT(V1)
      VA=LS*V1
      XP(2)=XP(2)+3.3
      XP(4)=XP(4)+(2.*D)
      MB(1)=XP(1)*XX(1,1)+XP(2)*XX(2,1)+XP(3)*XX(3,1)+XP(4)*XX(4,1)
      MB(2)=XP(1)*XX(1,2)+XP(2)*XX(2,2)+XP(3)*XX(3,2)+XP(4)*XX(4,2)
      MB(3)=XP(1)*XX(1,3)+XP(2)*XX(2,3)+XP(3)*XX(3,3)+XP(4)*XX(4,3)
      MB(4)=XP(1)*XX(1,4)+XP(2)*XX(2,4)+XP(3)*XX(3,4)+XP(4)*XX(4,4)
      V2=MB(1)*XP(1)+MB(2)*XP(2)+MB(3)*XP(3)+MB(4)*XP(4)
      V2=SQRT(V2)
      VB=LS*V2
      XP(2)=XP(2)-3.3
      MC(1)=XP(1)*XX(1,1)+XP(2)*XX(2,1)+XP(3)*XX(3,1)+XP(4)*XX(4,1)
      MC(2)=XP(1)*XX(1,2)+XP(2)*XX(2,2)+XP(3)*XX(3,2)+XP(4)*XX(4,2)
      MC(3)=XP(1)*XX(1,3)+XP(2)*XX(2,3)+XP(3)*XX(3,3)+XP(4)*XX(4,3)
      MC(4)=XP(1)*XX(1,4)+XP(2)*XX(2,4)+XP(3)*XX(3,4)+XP(4)*XX(4,4)
      V3=MC(1)*XP(1)+MC(2)*XP(2)+MC(3)*XP(3)+MC(4)*XP(4)
      V3=SQRT(V3)
      VC=LS*V3
      XP(2)=XP(2)+3.3
      XP(4)=XP(4)-(2.*D)
      MD(1)=XP(1)*XX(1,1)+XP(2)*XX(2,1)+XP(3)*XX(3,1)+XP(4)*XX(4,1)
      MD(2)=XP(1)*XX(1,2)+XP(2)*XX(2,2)+XP(3)*XX(3,2)+XP(4)*XX(4,2)
      MD(3)=XP(1)*XX(1,3)+XP(2)*XX(2,3)+XP(3)*XX(3,3)+XP(4)*XX(4,3)
      MD(4)=XP(1)*XX(1,4)+XP(2)*XX(2,4)+XP(3)*XX(3,4)+XP(4)*XX(4,4)
      V4=MD(1)*XP(1)+MD(2)*XP(2)+MD(3)*XP(3)+MD(4)*XP(4)
      V4=SQRT(V4)
      VD=LS*V4
      XP(2)=XP(2)-1.65
      XP(4)=XP(4)+D
      IF(VA.GE.VB.AND.VA.GE.VC.AND.VA.GE.VD)GO TO 320
      IF(VB.GE.VA.AND.VB.GE.VC.AND.VB.GE.VD)GO TO 330
      IF(VC.GE.VA.AND.VC.GE.VB.AND.VC.GE.VD)GO TO 340
      YAH=VD
      GO TO 360
  320 YAH=VA
      GO TO 360
  330 YAH=VB
      GO TO 360
  340 YAH=VC
```

```
C  XP(2)=WATER CONTENT IN PERCENT
C  XP(3)=COMPACTIVE EFFORT IN NUMBER OF PASSES
C  XP(4)=EXPECTED DRY DENSITY IN POUNDS PER CUBIC FOOT
C  Q=EXPECTED UNCONFINED SHEAR STRENGTH IN POUNDS PER SQUARE INCH
C  D=EXPECTED VARIATION IN DRY DENSITY IN PCF
C  YAH=EXPECTED VARIATION IN UNCONFINED STRENGTH IN PSI
C  DIFD=MINIMUM EXPECTED DRY DENSITY IN PCF
C  DIFS=MINIMUM EXPECTED UNCONFINED SHEAR STRENGTH IN PSI
C
  360 DIFD=XP(4)-D
      DIFS=Q-YAH
      WRITE(5,2000)XP(2),XP(3),XP(4),Q,D,YAH,DIFD,DIFS
      XAH=XAH+.1
  105 CONTINUE
      GRAPH=GRAPH+1.
  120 CONTINUE
      STOP
  100 FORMAT(1H1,25X,*SHEEPSFOOT ROLLER - AS-COMPACTED*,//)
  500 FORMAT(8A10)
  900 FORMAT(4F15.9)
 1005 FORMAT(4F15.9)
 1010 FORMAT(3(4F15.9,/),4F15.9)
 1015 FORMAT(2F10.6)
 2000 FORMAT(8F8.3)
      END
C
C  THE DATA DECK FOLLOWS
C
.9125198       -.061213       -.061213       .0041982
2.315     2.679
16.994601      -2.057526      .5846169       .3530319
53.578         -.59805        .02974         -.39853
-.59805        .009897        .0001438       .004005
.02974         .0001438       .0008995       -.0003387
-.39853        .004005        -.0003387      .00303
2.60      8.8025
```

COMPUTER PROGRAM USED FOR STRENGTH VARIABILITY CALCULATIONS:RTR-AC

```
C  XD=VECTOR OF INDEPENDENT VARIABLES FOR DRY DENSITY
C  XXD=[X'X]^-1  FOR DRY DENSITY
C  LAMDAD=λ FOR DRY DENSITY
C  SD=SQUARE ROOT OF THE MEAN SQUARE ERROR FOR DRY DENSITY
C  XP=VECTOR OF INDEPENDENT VARIABLES FOR STRENGTH
C  XX=[X'X]^-1  FOR UNCONFINED COMPRESSIVE STRENGTH
C  BETA=CONSTANT AND VARIABLE COEFFICIENTS OF THE STRENGTH EQUATION
C  LAMDA=λ FOR UNCONFINED COMPRESSIVE STRENGTH
C  S=SQUARE ROOT OF THE MEAN SQUARE ERROR FOR STRENGTH
C
      REAL LAMDA,MA,MB,MC,MD,LS,LAMDAD,LDS
      DIMENSION XP(4),XX(4,4),BETA(4),MA(4),MB(4),MC(4),MD(4),XXD(3,3),X
     1DPXXA(3),XDPXXB(3),XD(3)
C
C  READ NECESSARY INFORMATION
C
      READ(1,900)XXD
      READ(1,1015)LAMDAD,SD
      READ (1,1005)BETA
      READ (1,1010)XX
      READ(1,1015)LAMDA,S
      LS=LAMDA*S
      LDS=LAMDAD*SD
      PRINT 100
      XD(1)=1.0
      XP(1)=1.0
      GRAPH=2.
      DO 120 K=1,15
      XP(3)=GRAPH
      XD(3)=GRAPH
      XAH=11.
      DO 105 J=1,29
      XD(2)=XAH
      XP(2)=XAH
C
C  2 ENDPOINT DENSITY VARIABILITY CALCULATION
C
      XD(2)=XD(2)-1.85
      XD(2)=XD(2)**2
      XDPXXA(1)=XD(1)*XXD(1,1)+XD(2)*XXD(2,1)+XD(3)*XXD(3,1)
      XDPXXA(2)=XD(1)*XXD(1,2)+XD(2)*XXD(2,2)+XD(3)*XXD(3,2)
      XDPXXA(3)=XD(1)*XXD(1,3)+XD(2)*XXD(2,3)+XD(3)*XXD(3,3)
      VDD1=XDPXXA(1)*XD(1)+XDPXXA(2)*XD(2)+XDPXXA(3)*XD(3)
      VDD1=SQRT(VDD1)
      D1=LDS*VDD1
      XD(2)=SQRT(XD(2))
      XD(2)=XD(2)+3.7
      XD(2)=XD(2)**2
      XDPXXB(1)=XD(1)*XXD(1,1)+XD(2)*XXD(2,1)+XD(3)*XXD(3,1)
      XDPXXB(2)=XD(1)*XXD(1,2)+XD(2)*XXD(2,2)+XD(3)*XXD(3,2)
      XDPXXB(3)=XD(1)*XXD(1,3)+XD(2)*XXD(2,3)+XD(3)*XXD(3,3)
      VDD2=XDPXXB(1)*XD(1)+XDPXXB(2)*XD(2)+XDPXXB(3)*XD(3)
      D2=LDS*VDD2
      VDD2=SQRT(VDD2)
      D2=LDS*VDD2
      XD(2)=SQRT(XD(2))
```

```
      XD(2)=XD(2)-1.85
      IF(D2.GE.D1)GO TO 301
      D=D1
      GO TO 302
  301 D=D2
C
C  REGRESSION EQUATION EVALUATIONS FOR DENSITY AND STRENGTH
C
  302 XP(4)=122.46-(.04763*(XP(2)**2))+(.169*XP(3))
      Q=XP(1)*BETA(1)+XP(2)*BETA(2)+XP(3)*BETA(3)+XP(4)*BETA(4)
C
C  4 ENDPOINT STRENGTH VARIABILITY CALCULATION
C
      XP(2)=XP(2)-1.85
      XP(4)=XP(4)-D
      MA(1)=XP(1)*XX(1,1)+XP(2)*XX(2,1)+XP(3)*XX(3,1)+XP(4)*XX(4,1)
      MA(2)=XP(1)*XX(1,2)+XP(2)*XX(2,2)+XP(3)*XX(3,2)+XP(4)*XX(4,2)
      MA(3)=XP(1)*XX(1,3)+XP(2)*XX(2,3)+XP(3)*XX(3,3)+XP(4)*XX(4,3)
      MA(4)=XP(1)*XX(1,4)+XP(2)*XX(2,4)+XP(3)*XX(3,4)+XP(4)*XX(4,4)
      V1=MA(1)*XP(1)+MA(2)*XP(2)+MA(3)*XP(3)+MA(4)*XP(4)
      V1=SQRT(V1)
      VA=LS*V1
      XP(2)=XP(2)+3.7
      XP(4)=XP(4)+(2.*D)
      MB(1)=XP(1)*XX(1,1)+XP(2)*XX(2,1)+XP(3)*XX(3,1)+XP(4)*XX(4,1)
      MB(2)=XP(1)*XX(1,2)+XP(2)*XX(2,2)+XP(3)*XX(3,2)+XP(4)*XX(4,2)
      MB(3)=XP(1)*XX(1,3)+XP(2)*XX(2,3)+XP(3)*XX(3,3)+XP(4)*XX(4,3)
      MB(4)=XP(1)*XX(1,4)+XP(2)*XX(2,4)+XP(3)*XX(3,4)+XP(4)*XX(4,4)
      V2=MB(1)*XP(1)+MB(2)*XP(2)+MB(3)*XP(3)+MB(4)*XP(4)
      V2=SQRT(V2)
      VB=LS*V2
      XP(2)=XP(2)-3.7
      MC(1)=XP(1)*XX(1,1)+XP(2)*XX(2,1)+XP(3)*XX(3,1)+XP(4)*XX(4,1)
      MC(2)=XP(1)*XX(1,2)+XP(2)*XX(2,2)+XP(3)*XX(3,2)+XP(4)*XX(4,2)
      MC(3)=XP(1)*XX(1,3)+XP(2)*XX(2,3)+XP(3)*XX(3,3)+XP(4)*XX(4,3)
      MC(4)=XP(1)*XX(1,4)+XP(2)*XX(2,4)+XP(3)*XX(3,4)+XP(4)*XX(4,4)
      V3=MC(1)*XP(1)+MC(2)*XP(2)+MC(3)*XP(3)+MC(4)*XP(4)
      V3=SQRT(V3)
      VC=LS*V3
      XP(2)=XP(2)+3.7
      XP(4)=XP(4)-(2.*D)
      MD(1)=XP(1)*XX(1,1)+XP(2)*XX(2,1)+XP(3)*XX(3,1)+XP(4)*XX(4,1)
      MD(2)=XP(1)*XX(1,2)+XP(2)*XX(2,2)+XP(3)*XX(3,2)+XP(4)*XX(4,2)
      MD(3)=XP(1)*XX(1,3)+XP(2)*XX(2,3)+XP(3)*XX(3,3)+XP(4)*XX(4,3)
      MD(4)=XP(1)*XX(1,4)+XP(2)*XX(2,4)+XP(3)*XX(3,4)+XP(4)*XX(4,4)
      V4=MD(1)*XP(1)+MD(2)*XP(2)+MD(3)*XP(3)+MD(4)*XP(4)
      V4=SQRT(V4)
      VD=LS*V4
      XP(2)=XP(2)-1.85
      XP(4)=XP(4)+D
      IF(VA.GE.VB.AND.VA.GE.VC.AND.VA.GE.VD)GO TO 320
      IF(VB.GE.VA.AND.VB.GE.VC.AND.VB.GE.VD)GO TO 330
      IF(VC.GE.VA.AND.VC.GE.VB.AND.VC.GE.VD)GO TO 340
      YAH=VD
      GO TO 360
  320 YAH=VA
      GO TO 360
  330 YAH=VB
```

```
      GO TO 360
  340 YAH=VC
C
C  XP(2)=WATER CONTENT IN PERCENT
C  XP(3)=COMPACTIVE EFFORT IN NUMBER OF PASSES
C  XP(4)=EXPECTED DRY DENSITY IN PCF
C  Q=EXPECTED UNCONFINED SHEAR STRENGTH IN PSI
C  D=EXPECTED VARIATION IN DRY DENSITY IN PCF
C  YAH=EXPECTED VARIATION IN UNCONFINED STRENGTH IN PSI
C  DIFD=MINIMUM EXPECTED DRY DENSITY IN PCF
C  DIFS=MINIMUM EXPECTED UNCONFINED SHEAR STRENGTH IN PSI
C
  360 DIFD=XP(4)-D
      DIFS=Q-YAH
      WRITE(5,2000)XP(2),XP(3),XP(4),Q,D,YAH,DIFD,DIFS
      XAH=XAH+.1
  105 CONTINUE
      GRAPH=GRAPH+1.
  120 CONTINUE
      STOP
  100 FORMAT(1H1,30X,*RUBBER TIRE ROLLER, AS-COMPACTED*,//)
  500 FORMAT(8A10)
  900 FORMAT(2(3F15.9,/),3F15.9)
 1005 FORMAT(4F15.9)
 1010 FORMAT(3(4F15.9,/),4F15.9)
 1015 FORMAT(2F10.6)
 2000 FORMAT(8F8.3)
      END
C
C  DATA DECK FOLLOWS
C
.36342         -.00137223     -.004575912
-.00137223     .00000592      .00000395
-.004575912    .00000395      .00046606
2.303     2.25026
51.9059        -2.22412       -.155373       .073393
72.37839       -.855707       .079052        -.5326558
-.855707       .0140576       -.0007512      .0057651
.079052        -.0007512      .0006625       -.0006465
-.5326558      .0057651       -.0006465      .003996
2.581     6.45788
```

COMPUTER PROGRAM USED FOR STRENGTH VARIABILITY CALCULATIONS:RTR-S

```
C  XD=VECTOR OF INDEPENDENT VARIABLES FOR DRY DENSITY
C  XXD=[X'X]-1  FOR DRY DENSITY
C  LAMDAD=λ FOR DRY DENSITY
C  SD=SQUARE ROOT OF THE MEAN SQUARE ERROR FOR DRY DENSITY
C  XP=VECTOR OF INDEPENDENT VARIABLES FOR STRENGTH
C  XX=[X'X]-1  FOR UNCONFINED COMPRESSIVE STRENGTH
C  BETA=CONSTANT AND VARIABLE COEFFICIENTS OF THE STRENGTH EQUATION
C  LAMDA=λ FOR UNCONFINED COMPRESSIVE STRENGTH
C  S=SQUARE ROOT OF THE MEAN SQUARE ERROR FOR STRENGTH
C
      REAL LAMDA,MA,MB,MC,MD,LS,LAMDAD,LDS
      DIMENSION XP(4),XX(4,4),BETA(4),MA(4),MB(4),MC(4),MD(4),XXD(3,3),X
     1DPXXA(3),XDPXXB(3),XD(3)
C
C  READ NECESSARY INFORMATION
C
      READ(1,900)XXD
      READ(1,1015)LAMDAD,SD
      READ (1,1005)BETA
      READ (1,1010)XX
      READ(1,1015)LAMDA,S
      LS=LAMDA*S
      LDS=LAMDAD*SD
      PRINT 100
      XD(1)=1.0
      XP(1)=1.0
      GRAPH=2.
      DO 120 K=1,15
      XP(3)=GRAPH
      XD(2)=GRAPH
      XAH=11.
      DO 105 J=1,29
      XP(2)=XAH
C
C  2 ENDPOINT DENSITY VARIABILITY CALCULATION
C
      XP(2)=XP(2)-1.85
      XD(3)=XP(2)*XD(2)
      XDPXXA(1)=XD(1)*XXD(1,1)+XD(2)*XXD(2,1)+XD(3)*XXD(3,1)
      XDPXXA(2)=XD(1)*XXD(1,2)+XD(2)*XXD(2,2)+XD(3)*XXD(3,2)
      XDPXXA(3)=XD(1)*XXD(1,3)+XD(2)*XXD(2,3)+XD(3)*XXD(3,3)
      VDD1=XDPXXA(1)*XD(1)+XDPXXA(2)*XD(2)+XDPXXA(3)*XD(3)
      VDD1=SQRT(VDD1)
      D1=LDS*VDD1
      XP(2)=XP(2)+3.7
      XD(3)=XP(2)*XD(2)
      XDPXXB(1)=XD(1)*XXD(1,1)+XD(2)*XXD(2,1)+XD(3)*XXD(3,1)
      XDPXXB(2)=XD(1)*XXD(1,2)+XD(2)*XXD(2,2)+XD(3)*XXD(3,2)
      XDPXXB(3)=XD(1)*XXD(1,3)+XD(2)*XXD(2,3)+XD(3)*XXD(3,3)
      VDD2=XDPXXB(1)*XD(1)+XDPXXB(2)*XD(2)+XDPXXB(3)*XD(3)
      VDD2=SQRT(VDD2)
      D2=LDS*VDD2
      XP(2)=XP(2)-1.85
      XD(3)=XP(2)*XD(2)
      IF(D2.GE.D1)GO TO 301
      D=D1
```

```
      GO TO 302
  301 D=D2
C
C  REGRESSION EQUATION EVALUATIONS FOR DENSITY AND STRENGTH
C
  302 XP(4)=110.75+(1.529*XP(3))-(.0958*(XP(2)*XP(3)))
      Q=XP(1)*BETA(1)+XP(2)*BETA(2)+XP(3)*BETA(3)+XP(4)*BETA(4)
C
C  4 ENDPOINT STRENGTH VARIABILITY CALCULATION
C
      XP(2)=XP(2)-1.85
      XP(4)=XP(4)-D
      MA(1)=XP(1)*XX(1,1)+XP(2)*XX(2,1)+XP(3)*XX(3,1)+XP(4)*XX(4,1)
      MA(2)=XP(1)*XX(1,2)+XP(2)*XX(2,2)+XP(3)*XX(3,2)+XP(4)*XX(4,2)
      MA(3)=XP(1)*XX(1,3)+XP(2)*XX(2,3)+XP(3)*XX(3,3)+XP(4)*XX(4,3)
      MA(4)=XP(1)*XX(1,4)+XP(2)*XX(2,4)+XP(3)*XX(3,4)+XP(4)*XX(4,4)
      V1=MA(1)*XP(1)+MA(2)*XP(2)+MA(3)*XP(3)+MA(4)*XP(4)
      V1=SQRT(V1)
      VA=LS*V1
      XP(2)=XP(2)+3.7
      XP(4)=XP(4)+(2.*D)
      MB(1)=XP(1)*XX(1,1)+XP(2)*XX(2,1)+XP(3)*XX(3,1)+XP(4)*XX(4,1)
      MB(2)=XP(1)*XX(1,2)+XP(2)*XX(2,2)+XP(3)*XX(3,2)+XP(4)*XX(4,2)
      MB(3)=XP(1)*XX(1,3)+XP(2)*XX(2,3)+XP(3)*XX(3,3)+XP(4)*XX(4,3)
      MB(4)=XP(1)*XX(1,4)+XP(2)*XX(2,4)+XP(3)*XX(3,4)+XP(4)*XX(4,4)
      V2=MB(1)*XP(1)+MB(2)*XP(2)+MB(3)*XP(3)+MB(4)*XP(4)
      V2=SQRT(V2)
      VB=LS*V2
      XP(2)=XP(2)-3.7
      MC(1)=XP(1)*XX(1,1)+XP(2)*XX(2,1)+XP(3)*XX(3,1)+XP(4)*XX(4,1)
      MC(2)=XP(1)*XX(1,2)+XP(2)*XX(2,2)+XP(3)*XX(3,2)+XP(4)*XX(4,2)
      MC(3)=XP(1)*XX(1,3)+XP(2)*XX(2,3)+XP(3)*XX(3,3)+XP(4)*XX(4,3)
      MC(4)=XP(1)*XX(1,4)+XP(2)*XX(2,4)+XP(3)*XX(3,4)+XP(4)*XX(4,4)
      V3=MC(1)*XP(1)+MC(2)*XP(2)+MC(3)*XP(3)+MC(4)*XP(4)
      V3=SQRT(V3)
      VC=LS*V3
      XP(2)=XP(2)+3.7
      XP(4)=XP(4)-(2.*D)
      MD(1)=XP(1)*XX(1,1)+XP(2)*XX(2,1)+XP(3)*XX(3,1)+XP(4)*XX(4,1)
      MD(2)=XP(1)*XX(1,2)+XP(2)*XX(2,2)+XP(3)*XX(3,2)+XP(4)*XX(4,2)
      MD(3)=XP(1)*XX(1,3)+XP(2)*XX(2,3)+XP(3)*XX(3,3)+XP(4)*XX(4,3)
      MD(4)=XP(1)*XX(1,4)+XP(2)*XX(2,4)+XP(3)*XX(3,4)+XP(4)*XX(4,4)
      V4=MD(1)*XP(1)+MD(2)*XP(2)+MD(3)*XP(3)+MD(4)*XP(4)
      V4=SQRT(V4)
      VD=LS*V4
      XP(2)=XP(2)-1.85
      XP(4)=XP(4)+D
      IF(VA.GE.VB.AND.VA.GE.VC.AND.VA.GE.VD)GO TO 320
      IF(VB.GE.VA.AND.VB.GE.VC.AND.VB.GE.VD)GO TO 330
      IF(VC.GE.VA.AND.VC.GE.VB.AND.VC.GE.VD)GO TO 340
      YAH=VD
      GO TO 360
  320 YAH=VA
      GO TO 360
  330 YAH=VB
      GO TO 360
  340 YAH=VC
  360 DIFD=XP(4)-D
```

```
      DIFS=Q-YAH
C
C  XP(2)=WATER CONTENT IN PERCENT
C  XP(3)=COMPACTIVE EFFORT IN NUMBER OF PASSES
C  XP(4)=EXPECTED DRY DENSITY IN POUNDS PER CUBIC FOOT
C  Q=EXPECTED UNCONFINED SHEAR STRENGTH IN PSI
C  D=EXPECTED VARIATION IN DRY DENSITY IN PCF
C  YAH=EXPECTED VARIATION IN STRENGTH IN PSI
C  DIFD=MINIMUM EXPECTED DRY DENSITY
C  DIFS=MINIMUM EXPECTED SHEAR STRENGTH IN PSE
C
      WRITE(5,2000)XP(2),XP(3),XP(4),Q,D,YAH,DIFD,DIFS
      XAH=XAH+.1
  105 CONTINUE
      GRAPH=GRAPH+1.
  120 CONTINUE
      STOP
  100 FORMAT(1H1,25X,*RUBBER TIRE ROLLER - SOAKED*,//)
  500 FORMAT(8A10)
  900 FORMAT(2(3F15.9,/),3F15.9)
 1005 FORMAT(4F15.9)
 1010 FORMAT(3(4F15.9,/),4F15.9)
 1015 FORMAT(2F10.6)
 2000 FORMAT(8F8.3)
      END
C
C  THE DATA DECK FOLLOWS
C
.1483435976    -.0117622918   -.00005594
-.0117622918   .0223159       -.001413648
-.00005594     -.00143648     .0000933
2.435     2.4167
-118.36854     1.1807168      .19490556      .96835445
119.9967       -1.12698       .075285        -.92995
-1.12698       .021278        -.0002556      .0073025
.075285        -.0002556      0.0015153      -.0007503
-.92995        .0073025       -.0007503      .00741
2.7745    2.3234
```

COVER DESIGN BY ALDO GIORGINI